ROOF-TRUSS DESIGN

(PART 1)

CONDITIONS INFLUENCING DESIGN

1. Loads.—Roof trusses are especially adapted to such buildings as railroad terminals, opera houses, assembly halls, markets, etc., since they support the roof and still afford a clear space without the use of intermediate columns. They support loads of various kinds, such as the dead load, due to the weight of material used in constructing and covering the roof, the wind load, the snow load, and frequently the weight of plastered ceilings, as well as loads from attic floors and from suspended platforms and galleries.

The building laws of some cities require that trusses having a pitch, or slope, of less than 25° be designed to sustain a load of 50 pounds per square foot of actual roof area, and those of greater pitch to support a load of 30 pounds per square foot of area covered. In these values, ample allowance is made for snow and wind loads, in addition to the weight of material used for roof covering and construction. In laying out the stress diagrams, all these loads are considered, and both members and details are then proportioned to withstand the various stresses produced.

In supporting the roof covering, it is customary to introduce a secondary construction, which transfers the weight to the trusses. This usually consists of large beams, called *purlins*, that rest on and connect the trusses at their *panel points*, or points at which the main rafter and the web members intersect. These purlins support the rafters of the

roof, on which the wood sheathing and roofing material are placed. In wood construction, a purlin usually consists of a single rectangular timber hung or placed on top of the trusses, or mortised or gained into the cheek of the rafter member, while in iron and steel construction **I** beams and **Z** bars are generally employed.

Great care should be taken to avoid, as far as possible, the accidental stresses to which trusses are so often subjected during erection, and which can scarcely be calculated.

2. Slope of Roof.—The character of the roofing material must be considered in determining the direction of the upper chord. For instance, when shingles are used, the pitch should not be less than 1 of rise to 2 of run, while with slate, if the pitch is less than 1 to 3, the wind is liable to blow the rain under the slate, thereby causing leaks. A slope of 1 to 2, however, is preferable for slate, although, when occasion requires, the minimum pitch of 1 to 3 may be employed.

Corrugated iron is liable to leak if laid with a pitch of less than 1 to 3, while in a gravel and tar roof, if the slope is greater than 1 to 4, the heated tar is apt to run down and collect at the lower portion of the roof, leaving the upper part exposed and unprotected, and rain falling on such a roof flows off so quickly that the pebbles are washed out of the roofing. Flat clay tile set in asphalt may be used on flat roofs, but clay and metal tile simulating corrugated or Spanish tile are usually laid with a pitch somewhat greater than 1 to 2.

3. Distance Between Trusses.—The fact that the economy of the design is so largely dependent on the spacing of the trusses, makes it necessary that an effort be made to ascertain the distance that may most economically exist between them. This is especially the case when the building over which they are to be placed is of such a character that the spacing of the trusses governs, or, at least, affects the exterior design. Often, too, the engineer is restricted by the fact that the size of the lot must be considered when determining the size of the building, and hence, the distance

between the trusses is influenced to a certain degree, particularly when architectural effect is desired.

4. When the architectural design of the building is subordinate to the structural design, the distance between trusses may be most economical. This spacing is, however, somewhat difficult to determine, and no general rule can be given that holds true for specific cases. Extensive investigations, in which the material required and the cost of labor in both shop and field were considered, show that the most economical distance between trusses with spans of 40 to 200 feet is about one-fourth the span. Roof principals or trusses are seldom placed less than 10 feet and never more than 50 feet on centers. The usual span of the purlins for triangular trusses is from 12 to 20 feet, while arched trusses built in pairs and having purlin members in the form of latticed girders are seldom placed less than 25 feet or more than 50 feet apart.

5. Material Used.—The general design of a truss is influenced by the material employed in its construction, and the choice of material is influenced by its cost and availability, as well as by the span of the truss and the loads that come on it. When the span exceeds 80 feet, and the loads are comparatively heavy, steel is usually the best material to use; but if steel is unavailable, timber may be used for trusses having spans as great as 150 feet. When timber trusses have as great a span as this, they are usually arched, and built in pairs.

While steel may be used for the construction of trusses of all spans, great and small, the designer is frequently compelled to use timber because the cost of the work is limited. Timber trusses may be built entirely of wood, with the exception of the spikes and bolts used at the connections; they are adapted to localities far removed from industrial centers, where castings, special forgings, and steel shapes cannot readily be obtained. Ordinary timber trusses are built with wooden rafter and tie-members and struts, while the tension members extending from the foot of the struts are wrought iron or steel. When all the tension members

of a truss, including the tie-member, are steel or wrought iron, and the connections are made by pins, the type is known as a *composite truss*. In some composite trusses only the rafter members are timber, the struts being made of structural steel shapes latticed or tied together by clips.

6. After the general dimensions of the roof truss have been determined, if economy is to be considered, the cost must be investigated. Should the conditions admit a choice of several designs, it is often desirable to estimate the cost of each, and adopt the one whose construction costs the least.

In designing trusses it is generally cheaper to use stock sizes of timber or steel shapes, for by so doing, even though the members are of a larger size than actually required, the work is usually facilitated to such an extent that the time required in its performance is materially reduced, which is frequently a factor of the utmost importance. But if time need not be considered, and the work involves the use of large quantities of material, the design may usually be cheapened by using special sizes of timber or rolled shapes. The saving is due to the fact that the members of the frame may be proportioned with more exactness to withstand the stresses to which they are subjected, and thereby much less material is required. In most cases, however, the construction is not of sufficient magnitude to warrant the use of special sizes in either timber or rolled shapes, and the general practice is to give preference to stock sizes, using as small a variety as possible. Likewise, when the truss is being assembled and erected, labor is saved and complications are avoided in both shop and field by using, whenever practical, the same sized rivets, pins, and bolts throughout the frame.

TRUSS FORMS

7. King-Post Truss.—The simplest form of roof truss, as shown in Fig. 1 (*a*), is a triangular frame consisting of two equal rafter members connected by a tie-beam. The tie-beam becomes necessary when the outward thrust of the rafters is too great for the resistance of the walls. The

bending moment on the tension member *a b*, due to its own weight and the load of the ceiling, increases as the span of the truss increases. The section required to resist both the tensile and transverse bending stresses would necessarily be large; hence, to reduce the size of this member a suspension rod is introduced at *c d*, Fig. 1 (*b*), which makes the effective span of the tie-beam *a b* equal to one-half the span of the truss. For this reason the span of the truss shown in (*a*) is limited to about 24 feet, while the construction shown in (*b*) may be used with spans as great as 35 feet. When the span

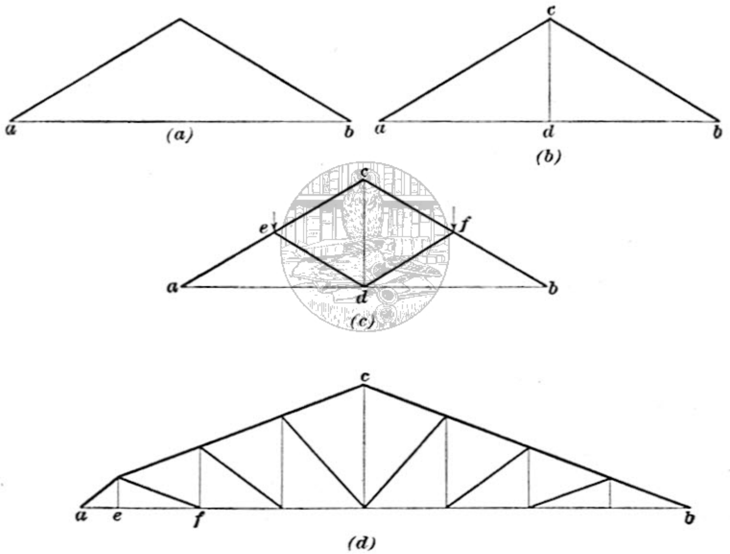


FIG. 1

is still further increased, it becomes desirable to introduce intermediate purlins at *e*, *f*, Fig. 1 (*c*). If these purlins were unsupported, a bending moment would be created on the rafter member at *e* and *f*; intermediate struts are therefore introduced at *e d* and *f d* to relieve the members of the transverse stress. Originally, the member *c d*, Fig. 1 (*c*), was made of wood and was known as a *king post*, or *king rod*, and the truss termed a **king-post truss.** But as there was some difficulty in making the connections, the form has

been modified by using a wrought-iron tension rod at cd. This form of truss may be adopted for frames having spans as great as 40 feet, but it is often found economical to employ the intermediate strut in triangular trusses of shorter span. Extra labor and material are necessitated by the introduction of the strut, but the reduction permissible in the size of the rafter member more than compensates for it.

8. Howe Truss.—The method of increasing the number of triangles, as shown in Fig. 1 (a), (b), and (c), may go on indefinitely and the natural outcome is the **Howe truss,** or **king- and queen-post truss,** as it is sometimes called. This truss, which is shown in Fig. 1 (d), may be extended to very large spans by increasing the number of panels, but it is not suitable for steep roofs, because as the span increases, the length of the struts toward the center becomes so great as to require timbers of large size. For long trusses the usual rise is one-seventh of the span. With long spans, it is customary to reduce the shear at the heel of the truss by placing the compression member nearest the wall more nearly vertical, as is shown at a, Fig. 1 (d). In this frame the vertical tie at e is not needed to resist any stress created in the frame, but is necessary to support the long extent of lower chord between f and a.

9. Queen-Post Truss.—The style of truss shown in Fig. 2 (a) is known as the **queen-post truss.** Its structural design is not as correct as the king-post, but it is very useful where a rectangular space is desired in the center of the room, as in an attic, or small hall.

When loaded unsymmetrically, it tends to assume the shape shown by the dotted lines; for this reason its use should be confined to cases where the loads are small and symmetrically placed. The whole tendency to resist an unsymmetrical load is directed in bending the tie-member ab at the points d, d'.

From Fig. 2 (b) it may be seen that the development of the queen-post truss is not as extensive as that of the king-post, although it may be used for larger spans when it is

cross-braced in the center, as shown by the dotted lines, but this defeats the primary use of the truss. When used without cross-bracing it should be strongly bracketed or braced with wrought-iron plates or knees at the angles of the rectangle.

10. Bow-String Truss.—Another form of truss commonly built of wood, and used for spans as large as 100 feet, is shown in Fig. 2 (*c*); this is known as the **bow-string truss.** Its upper chord may be either an arc of a circle or a parabola. The peculiarity of this design is that the intermediate cross-bracing under a vertical load sustains little or

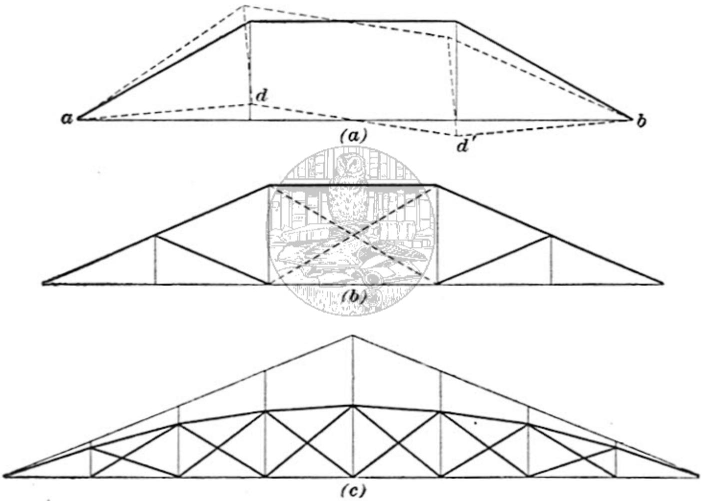


FIG. 2

no stress, according to the accuracy with which the curve of the upper chord coincides with the line of pressure of the loads. When these curves coincide, the cross-bracing serves to stiffen the design and to resist the strains due to eccentric and wind loads. When properly designed, the upper and lower chords take up all stresses from the vertical loads.

In order to utilize this form of truss, it is customary to extend the vertical members upwards to the required pitch line of the roof. By this means a roof of even pitch from apex to eaves may be supported.

11. Pratt Truss.—The truss shown in Fig. 3 (*a*) is known as the **Pratt;** its similarity to the Howe truss, Fig. 3 (*b*), can easily be seen, and the points of difference readily noted. In the Pratt truss, the compression members of the web of the frame, or the struts *a*, *a* are vertical, while the tension rods *b*, *b* are oblique. In the Howe, a reversed condition exists, for the struts are oblique and the tension members vertical. The Pratt offers rather a better appearance than the Howe from the fact that the oblique members, which usually extend at different angles, are round bars that are hardly noticeable, but in the Howe they are made of timbers, which are frequently unnecessarily heavy and far from pleasing in appearance, while the vertical timber struts

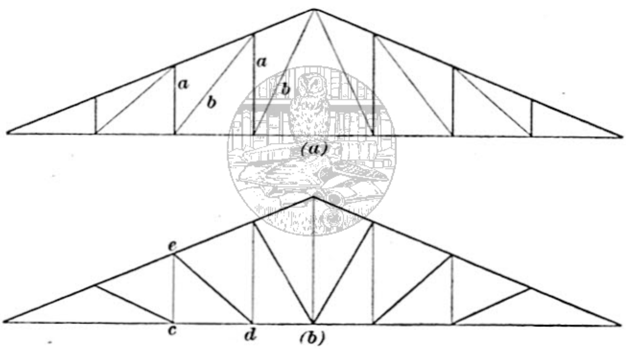


FIG. 3

used in the Pratt are entirely unobjectionable. Also, in the Pratt truss the compression members, or struts, are shorter than in the Howe, which is of advantage from the fact that while a short strut of a given section, as *a*, Fig. 3 (*a*), need be figured for direct compression only, a longer strut of the same section, as *e d*, Fig. 3 (*b*), is more apt to bend, and consequently must sometimes be figured to withstand both bending and compressive stresses. The Howe truss, however, has the advantage that the connections at *c*, *d*, and *e* are more conveniently made.

12. Church Roof Trusses.—The form of **church roof trusses** is generally greatly influenced by the architectural

treatment of the interior. The design usually includes a vaulted or arched ceiling, and the lower chord of the truss must be so arranged as to permit this effect. Most church roofs have a pitch of from 45° to 60°, which gives sufficient height for the arching or raising of the lower chord and also produces less outward thrust on the walls. From the fact that the walls of such edifices are usually buttressed and capable of resisting considerable thrust, the lower chord of the truss is relieved of much of the stress.

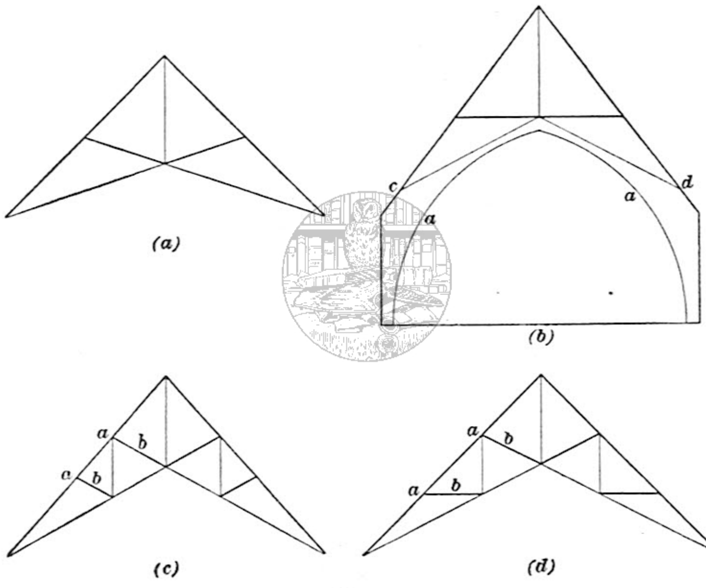


FIG. 4

The simplest type of church roof truss is known as the **scissors truss**; this, in its elementary form, is shown in Fig. 4 (*a*). It is a development of the king-post truss, with an intermediate strut, in which the lower member has been raised until the strut and the lower chord are in line. This truss, on account of its shape, is not suitable for large spans. A slight movement at the abutments or walls produces considerable distortion in the frame, greatly increasing the stress in the members.

Another form of the scissors truss is shown in Fig. 4 (*b*), in which the line of the vaulted ceiling is designated at *a a*. It is usual in planning ceilings of this character to form molded and projecting ribs beneath each truss, as such a treatment destroys the monotony of a plain ceiling and permits the lowering of the bottom chord or the tie-members of the truss.

Trusses built as shown in Fig. 4 (*b*) are apt to be weak at the connections *c* and *d*, because at these points there exists not only a compression in the extension of the rafter members, but a bending moment as well, due to the fact that the reactions about these points act with a lever arm equal to the perpendicular distance from their line of action to the point *c*. As this bending moment must be considered in designing the truss, and it is sometimes provided for with difficulty, it is best to avoid it, if possible, by connecting the tension members with the ends of the rafter members, as shown in Fig. 4 (*a*).

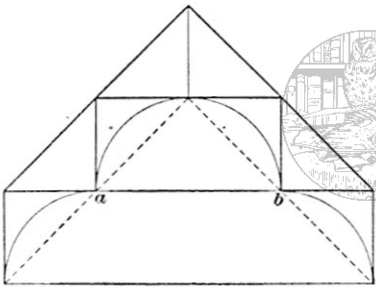


FIG. 5

With the increase of span, the scissors truss develops into the form shown in Fig. 4 (*c*) and (*d*). These types are satisfactory designs, and are applicable to spans as large as 60 feet. The intermediate purlins supporting the roof are located on the rafter member of the truss at the points *a*, *a*, the load being transmitted directly to the truss by means of the struts *b*, *b*.

In designing a truss of this character, it is always advisable to divide the upper chord into equal parts, because the appearance is more pleasing and the loads more uniform.

13. Hammer-Beam Truss.—A form of truss commonly used in church construction, shown in Fig. 5, is known as the **hammer-beam truss**. The lower chord is made up of curved pieces, which must be considered as

straight lines in solving the stress diagram. The form of the truss indicated by the dotted lines is assumed in the calculation, and the curved members are figured for both the tension and bending stress due to their being bent out of line.

Frequently trusses of this character are provided with a horizontal tie-member, extending from *a* to *b*. This member strengthens the frame considerably, and, when so constructed, these trusses may be used for spans up to 500 feet.

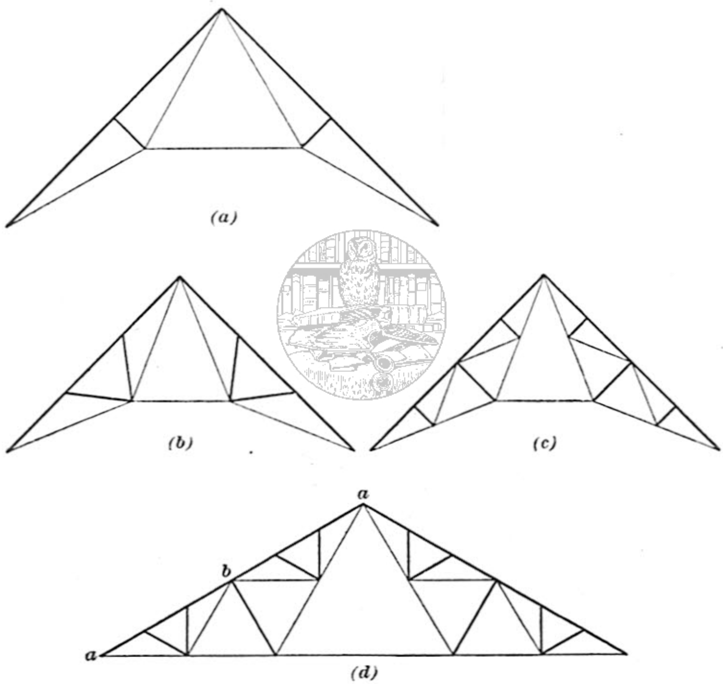


FIG. 6

14. Fink Truss.—The truss shown in Fig. 6 (*a*) is known as the **Fink truss,** from the engineer who first evolved the design. It is used frequently in roof construction, and on analysis may be considered as two trussed beams used as rafters, and connected at the lower ends of the struts by a tension rod or tie. The most common of the various types of Fink truss is provided with a straight tie or

lower chord, as shown in Fig. 6 (*d*). In order to gain headroom or height between the lower chord and the floor, the central member may be raised, as shown in (*a*), (*b*), and (*c*). The type in (*d*) may be used for spans as great as 100 or 125 feet. The main rafter member is supported between the points *a* and *b* by two secondary members instead of by one, as in the truss shown in (*c*).

The Fink truss may be of composite build, or may consist entirely of steel, but it is not adapted to construction entirely of wood. When made wholly of steel it is more economical for long spans than either the Howe or the Pratt, especially where the roof loads are light and there is no ceiling load, as in shops, car barns, etc.

15. Various Forms.—In addition to the forms of roof trusses described, many types are in use; a few of these are illustrated in Fig. 7. The form in (*a*) is a Howe truss that makes a frame of good appearance and provides increased headroom by having the lower chord raised at the center.

Gambrel roofs with spans varying from 40 to 60 feet are sometimes supported by trusses of the form shown in Fig. 7 (*b*). The suspension support for a flat ceiling is shown by the dotted lines. Trusses whose upper or compression chord is made up of several members extending in different directions from the various panel points, are of interest from the fact that the upper chord frequently approaches the form that would be assumed by the funicular or equilibrium polygon, and, consequently, where the lower chord is horizontal, the stresses in the web members *a*, *a* are eliminated when the truss is symmetrically loaded.

In Fig. 7 (*c*) is shown a quadrilateral truss built on the lines of a Howe, and supporting on its upper chord light scantling work for the sloping roof. A truss of this design can be used for large spans, and is particularly adapted for roofs over large halls, shops, shipways, and docks. Counterbraces should be provided, at least in the panels *a*, *a*, to prevent any distortion liable to be produced in the frame by unequally distributed snow loads and wind loads, although

they are not absolutely required if all the members in the truss are capable of resisting both tension and compression.

Roof covering for seats in the open, as at ball parks, tracks, and other outdoor places of amusement, may be constructed as shown in Fig. 7 (*d*). In this case the overhanging portion of the frame supports the roof over the front portion of the staging, where the choice seats are located. The overhanging, or cantilever, roof framing provides a support

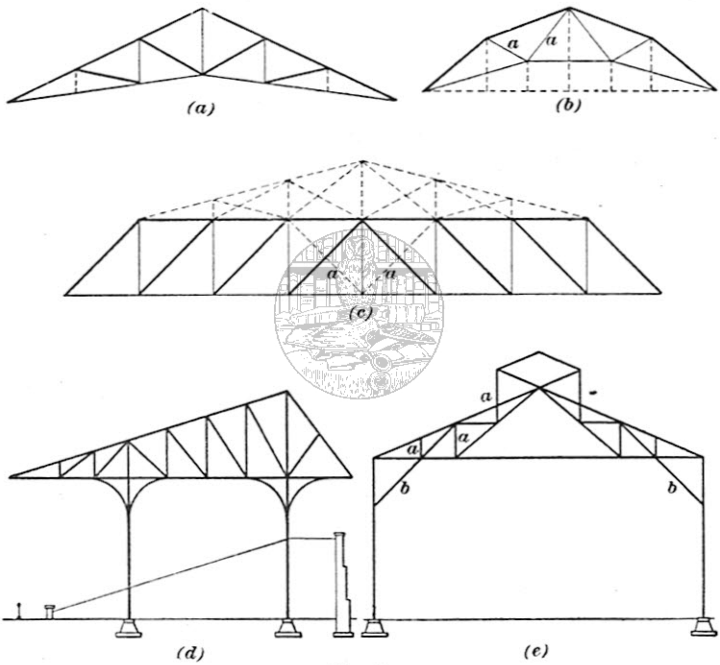


Fig. 7

for the covering without introducing columns in front, which would obstruct the view of the spectators. The columns supporting such trusses must be strong enough to resist the bending stress transmitted to them from the truss by the curved knee braces.

In this frame diagram no attempt has been made to designate the tension and compression members by light and heavy lines, since the character of the stress changes with the

different conditions of loading to which the truss is subjected. The stresses in the upper and lower chords of the cantilever are simply the reverse of those in the upper and lower chords of that portion of the frame between the columns, and all these stresses are subject to change when the wind is considered as blowing under the roof with a lifting action.

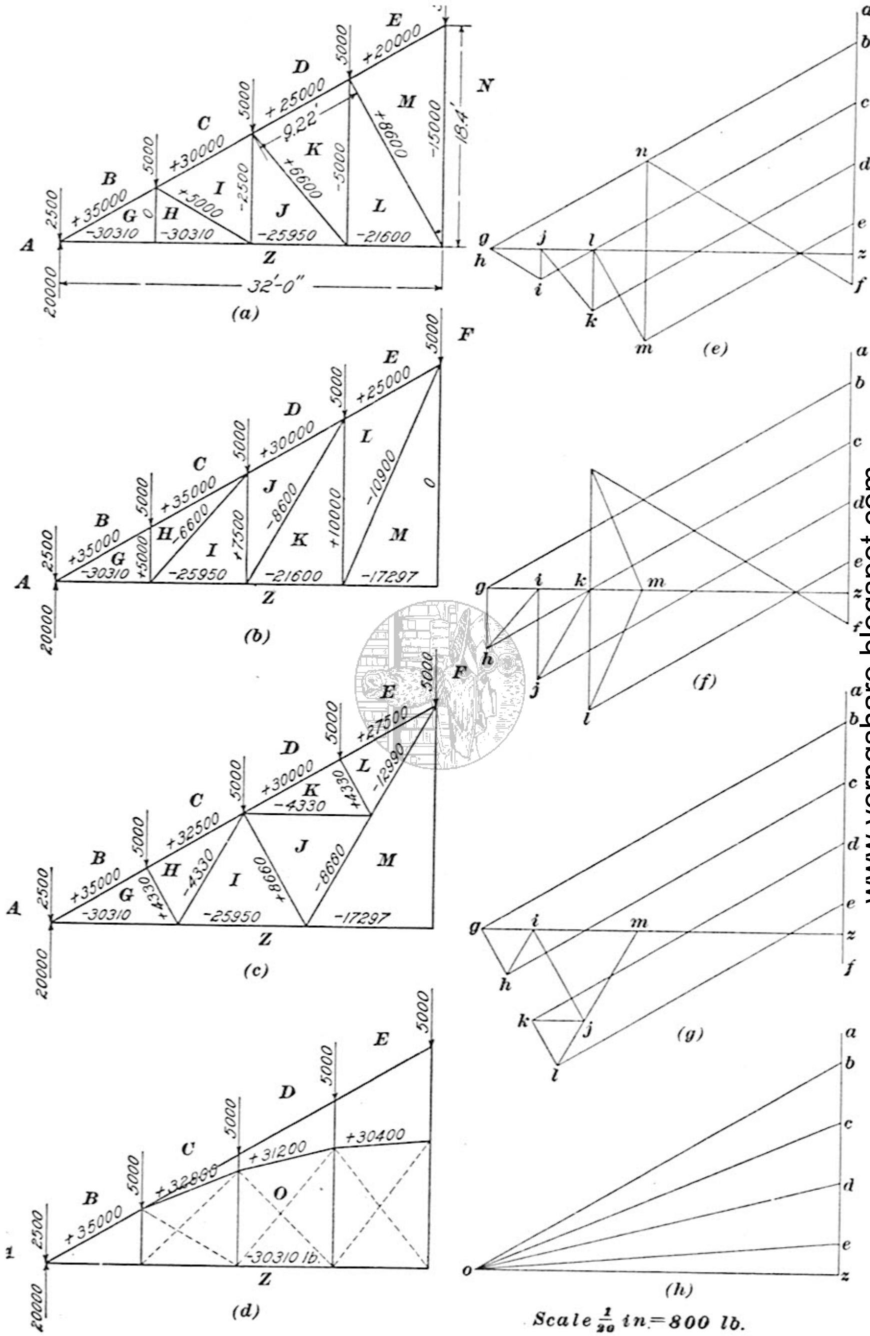
The modified Fink truss shown in Fig. 7 (*e*) is much used for mill buildings and shops. The struts *a*, *a* are arranged vertically instead of normal to the slope, and knee braces are provided at *b*, *b* to render the frame more rigid and to act as a wind brace, one being in tension and the other in compression when the wind acts from either side.

It is not customary to provide the columns supporting the truss with heavy foundation piers and footings, nor to consider them as fixed at the ends, which is an important item in determining the stresses in the frame. Such columns must be proportioned to withstand a considerable bending moment, which is created by the lateral thrust imposed on them by the knee braces.

16. Comparison of Forms.—In Fig. 8 (*a*), (*b*), (*c*), and (*d*) are shown four styles of trusses that support roofs of the same pitch and sustain loads of the same size, each being provided with a tie-rod. These illustrations represent Howe, Pratt, Fink, and bow-string trusses, respectively. Considering the first two, the maximum stress in the rafter member and also in the horizontal tie-member in (*a*) are equal, respectively, to those in (*b*), but a marked difference exists in the amount of stress in the several web members of the two trusses, the Howe displaying a decided advantage over the Pratt. The latter, however, is superior to the Howe in that the diagonal LM is subjected to a tensile stress instead of compressive and the longest compression strut is KL, which is much shorter than the member ML in the Howe.

The advantages of the Fink truss, Fig. 8 (*c*), over the Howe and Pratt trusses are:

1. It makes a better appearance, which, when the truss is exposed to view, is a consideration of no small importance.



Scale $\frac{1}{20}$ in.=800 lb.

FIG. 8

2. The struts are comparatively short, and being normal to the slope of the rafter member, afford good square bearing.

3. The tension members are generally made of wrought iron or steel, in which case it is usual to provide turnbuckles by which the strain is equally distributed throughout the members, and any shrinkage that might take place in the timber members can be provided for by further tightening the tension members.

The disadvantages of the Fink truss are that it cannot be successfully built without special tension rods and turnbuckles, while timber is used for the rafter members and struts. Such material conditions are difficult to meet in some localities, especially those in newly settled districts that are far from rolling mills, so that for timber trusses the Howe and Pratt are the cheapest; but for structural steel trusses, the Fink is preferred, both on account of economy of design and facility of construction and detailing.

The stresses in these three trusses are readily obtained by measuring the various lines in the stress diagrams shown in Fig. 8 (*e*), (*f*), and (*g*).

The upper chord of the truss in Fig. 8 (*d*) was so described as to coincide with the equilibrium polygon created by the loads on the truss, so that theoretically no web members, vertical or diagonal, are required. This equilibrium polygon is obtained by drawing lines parallel to the force polygon in (*h*). Owing to the fact that the loads on trusses are liable to be rendered unsymmetrical by snow and wind loads on one side of the roof, it is deemed advisable to employ light web members to make the design practical. In a timber truss these members may consist of light vertical wooden struts and wrought-iron tension bars; but a steel truss of large span is frequently constructed, as in bridge work, with a latticed upper chord, vertical struts, and diagonal tension bars or rods, with pin connections. From the stresses noted on the several members in the figure, it will be observed that the stress in the members of the upper chord varies but little, while in the lower chord it is uniform throughout. This condition greatly facilitates the economic design of the truss.

STRESS AND MATERIALS

STRESS

17. Strength of Parts.—The members of a truss may be subjected to tension, compression, bending, or shear, and in some cases to more than one of these stresses. When possible to avoid it, members that are subjected to tension and compression alternately should not be called on to resist cross-bending also, since in such a case one of the primary objects of truss construction, that is, the reduction of cross-bending to direct tension and compression, will be defeated.

18. In proportioning a member for tensile stress, the following rule is applicable:

Rule.—*Divide the tension to be resisted by the allowable stress per square inch; the quotient will be the required net section of the member. To this add an area sufficient to allow for any cutting or notching of members that may be necessary.*

The amount to be added is given under the separate heads in designing. In the case of tension members, it is well to support the long pieces at several places in order to relieve them of the extra strain they must withstand in supporting their own weight. This does not apply so much to slanting or vertical ties as it does to the horizontal members.

In proportioning compression members made of wood, it is customary to assume one dimension of cross-section, and if made of structural steel shapes, the radius of gyration is assumed. The total compression in the member divided by the allowable unit stress gives the required area for the member. There are many values for the allowable compression per square inch of timber, but the ones adopted for this Section are given in *Materials of Structural Engineering*, Part 3.

19. Compression and Tension Members Under Cross-Bending.—All members, except vertical ties and struts, are to a certain extent subjected to cross-bending as

well as to direct stress. These bending moments occur particularly: (1) When members placed in a horizontal or inclined position are required to support their own weight; (2) when the neutral axis of the member does not pass through the center of gravity of the connection; (3) when the member must support loads between its panel points.

20. Obtaining Bending Moment of Truss.—Sometimes the cross-bending stress due to the weight of the member may be counterbalanced by placing the centers of the pins or end connections out of line with the neutral axis of the member. If this is not done the amount of stress due to the two causes should be calculated and an area that is sufficient to reduce this stress to the safe limit should be added. There are three methods of calculating the bending moment.

First Method.—The simplest method consists in assuming a depth for the member, then calculating a width sufficient to withstand the bending moment, and adding to the area obtained by multiplying the width by the depth, an amount sufficient to provide for the direct stress. This is illustrated in the following example:

EXAMPLE.—What size timber will be required for a vertical compression member 11 feet $2\frac{1}{4}$ inches long that is subjected to a uniform load of 10,800 pounds, and a direct compression of 90,000 pounds?

SOLUTION.—The bending moment on a beam uniformly loaded and supported on both ends may be determined by the formula $M = \frac{WL}{8}$; hence, in this case, $M = \frac{10,800 \times 11.1875}{8} = 15,103\frac{1}{8}$ ft.-lb.; or, multiplying by 12, 181,237.5 in.-lb. The modulus of rupture of yellow pine is 7,000 lb.; then, assuming a factor of safety of 6, the safe working value of this material is $7,000 \div 6 = 1,167$ lb. per sq. in. The required section modulus for the rafter member is found by dividing the bending moment, 181,237.5 in.-lb. by 1,167. The quotient obtained is 155.3. The section modulus of any rectangular beam is equal to $\frac{b\,d^2}{6}$, so that if the depth of the member is assumed to be 14 in. the width or breadth required can be obtained by substituting in the formula $S = \frac{b\,d^2}{6}$, or $155.3 = \frac{b(14)^2}{6}$, from which b is found to equal 4.75 in.

The direct compression on the rafter must be considered next. If the breadth of the member is assumed to be no less than one-fifteenth

the length, a value of 1,000 lb. per sq. in. may be adopted. 90,000 $\div$ 1,000 = 90 sq. in., the area required to resist this stress. As a depth of 14 in. has already been considered, the additional width required in the rafter will be 90 $\div$ 14, or, approximately, 6.4 in., which, added to the width already calculated, gives the theoretical size of the member as 6.4 + 4.75 = 11.15 in. As there is no commercial size manufactured that is 11.15 in. by 14 in. in section, a 12 in. by 14 in. may be used with advantage, since the extra section compensates for the part cut away. Ans.

Second Method.—In this case the size of the member required to resist the direct stress is found. The maximum fiber stress per square inch due to the bending moment is then calculated for a member of this size, and added to the direct stress per square inch. The dimensions of the member need not be changed unless the sum of these stresses is more than 25 per cent. greater than the maximum allowable fiber stress under a direct load, in which case a larger size member should be adopted and the stresses must be again calculated.

EXAMPLE.—What size timber will be required to resist a compression of 33,400 pounds and a center load of 500 pounds, providing its length is 12 feet?

SOLUTION.—To approximate the size required, 1,000 lb. per sq. in. is assumed as the allowable stress. Then the area required would be $\frac{33400}{1000}$, or 33.4, and a section 6 in. by 6 in. would be used; but if this were the case, the length would be twenty-four times the width, and hence the calculation must be done by using the formula. Assuming the least dimension of the timber as 6 in., the allowable compression per square inch is $s_a = 825 - .175\left(\frac{l}{d}\right)^2 = 825 - .175\left(\frac{144}{6}\right)^2 = 825 - 100.8 = 724$ lb.

Dividing the compression, 33,400, by this value, the required area of the timber is 33,400 $\div$ 724 = 46.1, which can be provided for by a $6'' \times 8''$ timber. The extreme fiber stress due to bending is $\dfrac{\frac{500 \times 144}{4}}{\frac{6 \times 8^2}{6}} = 281$ lb. Since the greatest allowable compression under direct loads is 724 lb. per sq. in., the greatest allowable stress under the combined stress is $724 \times 1\frac{1}{4} = 905$ lb. per sq. in. The stress due to end compression is 33,400 $\div$ 48 = 696 lb. The sum of 696 and 281, the maximum fiber stress due to bending moment, equals 977 lb., and since this exceeds the allowable limit, the size must be increased and the stress recalculated.

Third Method.—In the two methods given, the bending moment due to the direct load multiplied by the deflection of the member under a bending moment is neglected. In some cases this is very great, as in a compression member the external bending moments tend to force the member out of line, giving it an initial deflection. This deflection forms an arm into which the direct compression acts, tending to bend the member still more, and thus increase the maximum fiber stress. It is of greater importance that this be considered in compressive members than in tension members, since in the latter case it is counteracted by the tension.

The method of calculating by the *third method* is given by the following formula, in which

M_2 = bending moment at point of maximum deflection, from cross-bending external forces and from eccentricity of position of longitudinal loading;

d = maximum deflection of member from all causes acting simultaneously;

W_t = total direct loading on member, tension or compression;

M_3 = bending moment from direct loading W_t into its arm $d = W_t d$;

M = total bending moment $= M_2 + M_3$;

A = area of sections;

s_1 = unit stress on extreme fiber from bending alone at section of maximum bending moment; or of maximum deflection, as the case may be, in pounds per square inch;

l = length of member, in inches;

c = distance from neutral axis to extreme fiber on which stress from bending is s_1;

E = modulus of elasticity;

I = moment of inertia of cross-section;

s_2 = unit stress in member from direct loading, supposed to be uniformly distributed $= \frac{W_t}{A}$;

s = total maximum unit stress in extreme fiber $= s_1 + s_2$

The total bending moment is then equal to $M = M_1 \pm M_2$, in which the positive sign refers to members under compression, while the negative sign refers to members under tension. The value of s_1 may be calculated by the formula

$$s_1 = \frac{M_2\,c}{I \pm \frac{W_t\,l^2}{10\,E}} \qquad (1)$$

in which the negative sign is used for compression members and the positive sign for tension members.

All dimensions given above should be taken in inches and all the forces in pounds. The maximum fiber stress s on the member is equal to the stress s_1 plus the stress due to the direct load s_2. This is illustrated by the following examples:

EXAMPLE 1.—What will be the maximum fiber stress due to bending and direct load on a horizontal tension bar 6 inches by $1\frac{1}{2}$ inches, whose length between pin points is 20 feet, and on which there is a direct tension of 126,000 pounds?

SOLUTION.—The weight of the bar is taken as .28 lb. per cu. in.; entire weight is $6 \times 1\frac{1}{2} \times 240 \times .28 = 604.8$ lb. The bending moment, therefore, is equal to $\frac{W\,l}{8} = \frac{604.8 \times 20 \times 12}{8} = 18{,}144$ in.-lb. Since the beam is rectangular in section, c, or the distance from the neutral axis to the extreme edge, is equal to $\frac{h}{2} = \frac{6}{2}$, or 3 in. I, or the moment of inertia of a rectangular beam, is equal to $\frac{b\,h^3}{12}$, which in this case is $\frac{1\frac{1}{2} \times (6)^3}{12}$, or 27. E equals 28,000,000; W_t equals 126,000. Then,

$$s_1 = \frac{18{,}144 \times 3}{27 + \frac{126{,}000 \times (240)^2}{10 \times 28{,}000{,}000}} = \frac{54{,}432}{52.92} = 1{,}028 \text{ lb., approximately}$$

The direct stress due to tension $= s_2 = \frac{W_t}{A} = \frac{126{,}000}{6 \times 1\frac{1}{2}} = 14{,}000$. Hence, the maximum fiber stress is equal to the sum of these two loads, or $14{,}000 + 1{,}028 = 15{,}028$ lb. Ans.

EXAMPLE 2.—In the above example, what would be the maximum fiber stress if the first method were employed; that is, adding together the stress due to bending moment and that due to tension?

SOLUTION.—The stress due to bending moment equals 18,144, while that due to tension equals 126,000. $126{,}000 + 18{,}144 = 144{,}144$ lb.

$$s_2 = \frac{W_t}{A} = \frac{144{,}144}{1\frac{1}{2} \times 6} \approx 16{,}016 \text{ lb. Ans.}$$

EXAMPLE 3.—What is the difference in tensile strength between a $5'' \times 1''$ tension bar 20 feet long placed flat and the same bar placed edgewise, provided that the maximum stress is not to exceed 15,000 pounds per square inch?

SOLUTION.—The weight of the member is $5 \times 1 \times 240 \times .28 = 336$ lb. The bending moment is $\frac{336 \times 240}{8} = 10{,}080$ in.-lb.

$$s_1 = \frac{10{,}080\, c}{I + \frac{W_t\, l^2}{10\, E}};\quad \frac{W_t\, l^2}{10\, E} = \frac{75{,}000 \times 240 \times 240}{10 \times 28{,}000{,}000} = 15.4$$

In the first case, $c = \frac{1}{2}$ and $I = \frac{5 \times (1)^3}{12} = .4167$, making

$$s_1 = \frac{10{,}080 \times \frac{1}{2}}{.4167 + 15.4} = 319 \text{ lb.}$$

Then the tensile strength of the bar placed flat is $15{,}000 - 319 = 14{,}681$. $14{,}681 \times 5 = 73{,}405$ lb., the strength of the bar.

In the second case, $c = 2\frac{1}{2}$ and $I = \frac{1 \times 5^3}{12} = \frac{125}{12}$, or 10.42, making $s_1 = \frac{10{,}080 \times 2\frac{1}{2}}{10.42 + 15.4} = 976$ lb. Hence, the tensile strength of the bar per square inch, when placed edgewise, is $15{,}000 - 976 = 14{,}024$ lb., and $5 \times 14{,}024 = 70{,}120$ lb.

The difference, in favor of the bar placed flat, is therefore $73{,}405 - 70{,}120 = 3{,}285$ lb. Ans.

EXAMPLE 4.—What will be the maximum fiber stress on a horizontal pin-connected compression member composed of two 10-inch 15-pound channels 18 feet long, placed back to back, supporting a compressive load of 100,000 pounds?

SOLUTION.—In the formula $s_1 = \frac{M_2\, c}{I - \frac{W_t\, l^2}{10\, E}}$, $W = 15 \times 18 \times 2 = 540$ lb.; $M_2 = \frac{W\, l}{8} = \frac{540 \times 216}{8} = 14{,}580$ in.-lb.; I, as given in the handbooks, for one channel, is 66.82; therefore, for two channels it is 133.64. $\frac{W_t\, l^2}{10\, E} = \frac{100{,}000 \times 216 \times 216}{10 \times 28{,}000{,}000} = 16.66$; $c = \frac{h}{2} = \frac{10}{2} = 5$; $s_1 = \frac{14{,}580 \times 5}{133.64 - 16.66} = \frac{72{,}900}{116.98} = 623.183$; $s_2 = \frac{W_t}{A} = \frac{100{,}000}{8.8} = 11{,}363.636$;

$$s = s_1 + s_2 = 623.183 + 11{,}363.636 = 11{,}986.919 \text{ lb. Ans.}$$

EXAMPLE 5.—What will be the difference between the maximum fiber stress calculated by the first method, and that calculated by the third method for the above compression member with a concentrated load of 3,500 pounds at the center?

SOLUTION.—In this case the total bending moment on the compression member is equal to the bending moment due to the weight of the member plus that due to the external load of 3,500 pounds at the center, or $\frac{Wl}{8} + \frac{3{,}500\,l}{4} = 14{,}580 + \frac{3{,}500 \times 216}{4} = 14{,}580 + 189{,}000$ $= 203{,}580$ in.-lb.; $c = 5$ in., and $I - \frac{Pl^2}{10E} = 117$, approximately. Thus, $s_1 = \frac{203{,}580 \times 5}{117} = \frac{1{,}017{,}900}{117} = 8{,}700$; $s_2 = 11{,}363.636$, as before, $s_1 + s_2 = 8{,}700 + 11{,}363.64 = 20{,}063.64$ lb. maximum fiber stress.

The maximum fiber stress due to the bending moment, when considered as acting alone, is equal to the total bending moment as calculated, divided by the section modulus of the member. The section modulus for a 10-in. 15-lb. channel is 13.4, then for two channels it is 26.8. $203{,}580 \div 26.8 = 7{,}596.26$ lb. This, added to the direct compression, equals $7{,}596.26 + 11{,}363.64 = 18{,}959.9 = 18{,}960$. The difference in results of the two methods of calculation is $20{,}063.64 - 19{,}560 = 1{,}103.64$. Ans.

In many cases it is very important that the stress due to the bending moment be considered in connection with the compressive stress in the member, especially where the compression members are designed to carry concentrated loads.

MATERIALS OF CONSTRUCTION

21. Timber.—The materials usually employed in the construction of roof trusses are wood, cast iron, wrought iron, and steel. Steel is probably used most, but is sometimes so expensive that for the sake of economy, especially in non-fireproof buildings, it is well to substitute **timber.** Since such timber must be tough, strong, and durable the varieties available are necessarily limited. Georgia long-leaf yellow pine is the best that can be obtained, and is especially adapted to the construction of trusses on account of its uniform grade, as well as its high tensile and compressive strength. The Douglas, Oregon, and Washington fir, or pine, are also very strong and hence may be very advantageously employed in roof construction, while spruce, hemlock, and northern, or short-leaf, yellow pine are used to some extent, although hemlock, having a low tensile and compressive strength, is useful only in trusses of short span that are called on to

sustain but little stress. White pine is seldom specified for truss construction, as it is getting expensive and does not readily resist compression. White oak is excellent for keys, bearing blocks, pins, etc., because it is tough and strong, but care must be taken concerning its location, since it shrinks considerably in seasoning. It can readily be seen that all woods last longer when protected from the weather, and for this reason a smaller factor of safety may be assumed for wood employed in heated buildings and other positions where it is not readily affected by dampness and changes of temperature.

TABLE I

COMMERCIAL SIZES OF TIMBER

Inches	Inches	Inches	Inches
1 × 8	2 × 10	4 × 6	8 × 8
1 × 12	2 × 12	4 × 8	8 × 10
1 × 16	3 × 4	4 × 10	8 × 12
1 × 18	3 × 6	4 × 12	10 × 10
2 × 3	3 × 8	6 × 6	10 × 12
2 × 4	3 × 10	6 × 8	10 × 14
2 × 6	3 × 12	6 × 10	12 × 12
2 × 8	4 × 4	6 × 12	

NOTE.—The size 10 × 14 may be obtained, but is not usually kept in stock.

22. Timber is made in certain stock sizes, as indicated in Table I, and although it is possible to obtain odd sizes, they are usually more expensive, as they are made by cutting down the stock next larger in section. Then, too, some stock sizes are cheaper than others, so that in many instances a considerable saving may be effected by arranging the design so as to use these cheaper timbers.

23. Influence of Shrinkage and Warping in Timber.—Since the rigidity of joints and connections is largely dependent on the warping, shrinkage, and twisting of the

material employed in truss construction, these matters must be given the earnest and careful study of the designer.

No woods are exempt from swelling caused by atmospheric changes, nor from shrinkage, which occurs during the process of seasoning; the shrinkage usually amounts to $\frac{1}{4}$ or $\frac{3}{8}$ inch per foot of width for ordinary commercial timber employed in roof trusses and subjected to the drying influence of a heated interior. The shrinkage lengthwise of the material is but slight, and is exceeded to a considerable extent by that which takes place in the width and thickness. The woods most affected are hickory and oak—especially the red oak, those least affected being soft pine, spruce, cedar, and cypress, while hard pine shrinks more than soft pine. When bolts are used as a means of fastening, care must be observed that the thickness of cross timber through which they extend is reduced to a minimum, and that no fastening camber rod or steel or iron tie is used unless some means of adjustment, such as threaded ends with nuts and washers or turnbuckles, be provided to take up any shrinkage that may occur after the frame has been in place and subjected to the atmosphere of the building for some months.

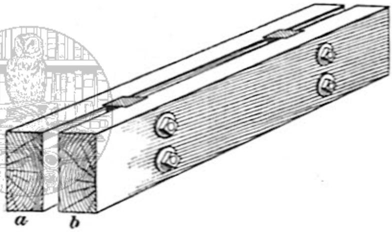


FIG. 9

All large-size timbers, particularly yellow pine, when not properly and completely seasoned, are liable to warp and twist, for which reason it is never advisable to use such material for important members, as the rigidity of the connection will be sacrificed and the strength decreased considerably. When there is a possibility of the material warping, important members may, with advantage, be built up of timbers 3 to 5 inches thick and kept about 2 inches apart by means of separators, as shown in Fig. 9. Since in such construction, members of comparative thinness are employed, it is possible to secure material whose quality is known, and the

tendency to warping can be modified by placing the timbers so that the grain is in the relative positions of *a* and *b*, Fig. 9.

24. Cast Iron.—Compression pieces in trusses were formerly made of cast iron, but owing to the fact that this material is not very reliable when subjected to changes of stress and of temperature, its use has been limited to bearing plates, heel connections, and small details.

25. Wrought Iron.—Wrought iron being the purest form of commercial iron, is more readily attacked by water and acids than either cast iron or steel. It is, however, a very tough, strong material of uniform texture, being used in truss construction for bolts, rivets, nuts, washers, and often for tie-rods, and is particularly adapted for use as tension members. At one time structural shapes were made of this material, but since steel, when manufactured by the modern process, is much cheaper and stronger than wrought iron, these rolled sections are now almost entirely made of steel.

26. Steel.—Steel is now employed to a great extent in construction of all kinds, and being so much stronger than wrought iron, the various pieces may be made considerably lighter. As in the case of timber, certain shapes and sizes are always kept in stock, by the use of which most designs may be executed with the expenditure of less time and money than when special sizes are ordered. The sizes manufactured by the various steel companies are given in the handbooks that they publish. Steel is made up in a number of shapes, such as angles, **I** beams, **Z** bars, channels, **T** bars, deck beams, flats, and rounds. Angles are most commonly used as truss members, since they are cheaper, can be more easily rolled, and can be handled with greater convenience than the other shapes. **I** beams and **Z** bars often serve as purlins, while in the larger trusses struts and beams are made by riveting pairs of channels together with plates and bars. Deck beams and **T** bars are not commonly employed in such construction, as similar shapes can be readily obtained by combining angles and plates. Rounds and flats

serve as tension members, especially in composite and pin-connected trusses. The pound price of steel is practically uniform for the usual rolled shapes, but channels and **T** bars are ordinarily more expensive than the others.

FACTOR OF SAFETY

27. Since due allowance must be made for unforeseen and unknown defects in materials and workmanship, and for unknown stresses that are liable to occur, the several parts of a structure must be proportioned to safely resist forces much greater than those obtained from the stress diagram. However, the stresses in roof trusses can be calculated with greater certainty than those in bridges or machines, as their application is more steady in its nature, and therefore not so severe on the material. Hence, in the design of roof trusses, unit stresses are permissible that are 50 per cent. in excess of those considered allowable in first-class bridges.

28. To provide greater ease in calculation, the materials and their values that are to be used are given in the tables for strength of materials. The allowable stress per square inch of material can readily be obtained by dividing the ultimate strength per square inch, as given in the table, by the factor of safety to be employed.

The strength of tension members can be conveniently obtained, but that of long columns must be calculated by substituting in the formula, while the values for short columns may be taken from the tables. In allowing for compression per square inch perpendicular to the grain, it is customary to use such strength values that the indentation of the wood will not exceed $\frac{1}{100}$ inch. This has been taken into consideration in determining the compressive strength of yellow pine perpendicular to the grain, as given in the various tables.

TRUSS DESIGN

29. After it has been decided what form of truss is to be used, and the intensity of the stresses has been obtained from the stress diagrams, it is necessary to calculate the sizes of the compression and tension members, and to design the connection. If the truss is of timber the size of the main rafters is especially important, since they are the principal compression members, and on their section the entire design of the truss is largely dependent.

In detailing joints and connections, the rule that axial lines of all members shall intersect at a single center at each joint or connection should be observed in all possible instances, as deviation therefrom to any considerable extent will cause bending moments in some of the members large enough to necessitate an increase in their size. When the parts used are rectangular in form, whether composed of steel or of timber, the axial line coincides with the center line of the member, and, of course, passes through the center of gravity of each cross-section, but when rolled sections are used, as, for instance, two angle irons riveted together, the center of gravity of the section must be located and the axial line made to pass through it.

TIMBER-TRUSS DESIGN

MEMBERS

30. Compression Members.—In order to obtain the section required for a **compression member** its length and the stress to be resisted must be known. The length, which is the distance between panel points, can be obtained from the frame diagram of the truss by scaling.

In most wooden trusses the upper chord is made in one piece from heel to peak, the size being determined by the

greatest stress. There is no economy in making the upper chord of separate pieces proportioned to the stress in each panel, because the extra bolts and the increased amount of labor required in making the joints cost more than the additional material made necessary by having the upper chord in one piece. Then, too, it is impossible to make a joint in a timber that will be as rigid as a continuous piece. The quantity of material saved by exactly proportioning the upper chord for the stress between panel points is illustrated in Fig. 8 (*c*), where the difference between the compression in the first and in the last panel of the Fink truss shown is 7,500 pounds, which represents a resistance of only 7.5 square inches of timber section, if an allowable unit compressive value parallel with the grain of 1,000 pounds is assumed.

31. It is difficult to determine the section required for a compression member of a given length to withstand a certain stress. The following method of approximating its dimensions is recommended: Assume a value for the ratio between the length of the column and the least dimension of the section of the column, both in inches, or for $\frac{l}{d}$, which will be the usual value for timber columns or posts. In most columns or struts this value $\frac{l}{d}$ is not less than 10, and no timber should be used as a compression member whose length is more than forty-five times its least dimension. The formula most frequently used for determining the strength of timber columns or struts is as follows:

$$u = s_c - \left(\frac{s_c\, l}{100\, d}\right) \qquad (2)$$

in which u = ultimate compressive strength of strut per square inch of section;

s_c = ultimate compressive strength of material per square inch parallel to grain;

l = length of column, in inches;

d = dimension of least side of column or post, in inches.

When the size of the strut section is to be determined, a given value, say 18, may be used for $\frac{l}{d}$, and the value of u found by substituting in the formula as follows:

$$u = s_c - \left(\frac{s_c}{100} \times 18\right)$$

or

$$u = s_c - (.18\,s_c) = .82\,s_c \qquad (3)$$

For example, consider that the timber composing a strut supporting a load of 80,000 pounds has a safe unit resistance of 1,000 pounds per square inch, and that it is necessary to decide on the approximate sectional area of the strut before even the details can be designed, or the actual working resistance of the strut found. Substituting in formula **3**, $u = .82\,s_c$, or $.82 \times 1{,}000 = 820$ pounds. The total stress in the member, 80,000 pounds, divided by 820 pounds, the safe unit stress, gives a required sectional area of 97.56 square inches. Considering this column as square in section, the length of the side will be equal to the square root of this area, or 9.88 inches, which may be considered as 10 inches.

If a timber of square section is not desired, one side may be assumed and the other determined by dividing the approximate area required by the length of the assumed side. The general practice is to use timbers whose sectional dimensions do not differ by more than 1 or 2 inches for the smaller sizes, and by 3 or 4 inches for the larger. In case the dimensions calculated do not correspond with any commercial size of timber, the size next larger should be adopted. Even when the calculations require an exact size of timber, it is customary to adopt the one next larger to allow for the shrinkage, dressing, and cutting that the timber must sustain during its manufacture and erection.

32. In order that the application of the above formulas and calculations may be clearly understood, the following example is given:

EXAMPLE.—A strut 16 feet long is required to resist a compressive stress of 20,000 pounds. If the strut is built of yellow pine, having an

allowable unit compressive value parallel with the grain of 1,000 pounds, what size of square timber will be required?

SOLUTION.—The approximate size of the timber required for the strut may be determined by applying formula **3**, $u = .82\,s_c = 820$ lb. This result is the approximate safe unit stress, in pounds, of the column section, so that the area of column section required is $20{,}000 \div 820 = 24.39$ sq. in. As the strut is to be square in section, the approximate dimension of the side of the strut will be $\sqrt{24.39} = 4.94$ in., or nearly 5 in. The commercial size of yellow pine of square section nearest to this dimension is 6 in. by 6 in., which when dressed will be reduced to about $5\frac{3}{4}$ in. by $5\frac{3}{4}$ in. Considering the strut as a wooden column $5\frac{3}{4}$ in. square and 16 ft., or 192 in., long, the approximate size having been calculated, it is now necessary to determine whether the strut will safely sustain the imposed load of 20,000 lb. The desired result is obtained by applying formula **2**, in which $u = s_c - \left(\frac{s_c\,l}{100\,d}\right)$; by substituting,

$$u = 1{,}000 - \left(\frac{1{,}000 \times 192}{100 \times 5.75}\right) = 667 \text{ lb.}$$

The sectional area of a strut 5.75 in. square is 33.06 sq. in., and if 667 lb. is the allowable unit compressive value of the strut, its entire safe resistance is $33.06 \times 667 = 22{,}051$ lb. From this result it is evident that a strut made of $6'' \times 6''$ timber dressed on four sides will be strong enough. Ans.

33. Although it is not customary to splice the upper chord of a timber truss, it is sometimes done in Howe trusses where the difference in stress throughout the panel section is great. The construction commonly adopted in this case is shown in Fig. 10, where a timber of uniform size, large enough to resist the stress in two of the upper panel sections of the truss, is used from peak to heel. The increased stress in the two lower panels is resisted by a separate piece of timber, which is securely bolted to the rafter member and into which the struts a, a are butted or framed. In explanation of this: The size of the member required for the upper chord is determined by the stress in the member DK, the excess of the stress in BG being resisted by the reenforcing piece $b\,b$.

EXAMPLE.—What size timbers will be required for the rafter member in the Howe truss shown in Fig. 8 (*a*) if the main rafter member is reenforced through the two lower panels?

Solution.—The short strut, or reenforcing piece, is required to extend through the first two panel points. The greatest compression on the two upper panels of the main rafter member is 25,000 lb., so that this stress determines the size of the member. The truss is assumed to be built of yellow pine having an allowable unit compressive stress of 1,000 lb., and the approximate size of the timber may be determined by formula **3**. By the application of this formula, it is found that the approximate sectional area required is equal to 30.48 sq. in. To attain this sectional area it is necessary to use a timber that, when dressed, will not be less than $5\frac{1}{2}$ in. by $5\frac{1}{2}$ in. Table I shows that the nearest commercial size to $5\frac{1}{2}$ in. by $5\frac{1}{2}$ in. is 6 in. by 6 in., which will be reduced, by dressing, to $5\frac{3}{4}$ in. by $5\frac{3}{4}$ in. The

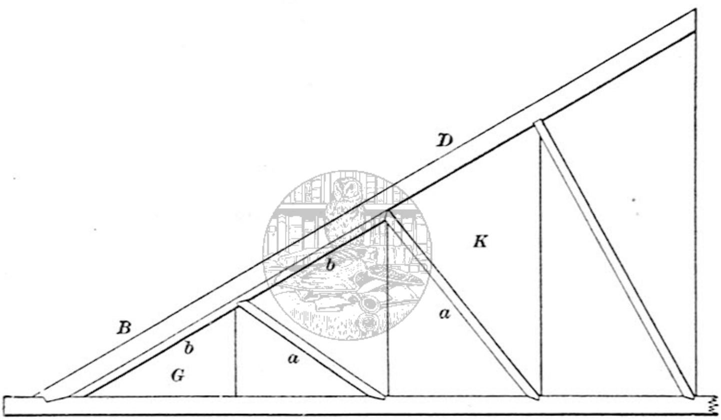


Fig. 10

unsupported length of the rafter member between panel points is 9.22 ft., or 110.64 in., so that by applying formula **2**, the value $u = 1{,}000 - \left(\frac{1{,}000}{100} \times \frac{110.64}{5.75}\right) = 808$ lb. The area of the rafter member being 5.75 in. by 5.75 in. = 33.06 sq. in., the safe resistance of the piece is $33.06 \times 808 = 26{,}712$ lb. The greatest stress in the rafter member is in the panel section adjacent to the heel of the truss and is equal to 35,000 lb. A resistance equal to 26,712 lb. is provided by the principal timber, so that a reenforcing piece provided through the two lower panels will be required to sustain a stress of $35{,}000 - 26{,}712 = 8{,}288$ lb. Assuming that because of such practical considerations as notching for the framing of the struts, this timber must be at least 4 in. in depth, and as it must be of the same width as the main timber, its minimum size is 4 in. by 6 in., which is reduced by dressing to $3\frac{3}{4}$ in. by $5\frac{3}{4}$ in. The area of a timber of this size is 21.56 sq. in., so that if it has a safe unit stress of 400 lb., its total resistance will equal

8,624 lb., which is more than sufficient, since a stress of but 8,288 lb. must be resisted.

To determine whether the reenforcing piece possesses a safe resistance of at least 400 lb. per sq. in., it is necessary to apply formula **2**, taking for the value d, the dimension of the least side, $3\frac{3}{4}$ in. Substituting in this formula, the value u is found as follows: $u = 1{,}000 - \left(\frac{1{,}000}{100} \times \frac{110.64}{3.75}\right) = 705$ lb. The result of this calculation demonstrates that the reenforcing piece is much stronger than is required, though, for practical features of construction, it would not be advisable to use a timber of smaller size. Ans.

34. Although the method just described is usually of no practical value in saving material, it gives a simple means of making the last two strut joints, and when employed, the strength of the small pieces should be carefully figured by the principles illustrated. Frequently, owing to the unavailability of material of the proper size, the rafter member of a timber truss must be reenforced, in which case these principles for calculation may be advantageously applied.

35. Tension Members.—The size of a tension member is found by dividing the tensile stress to be resisted by the safe unit tensile strength of the material; the result will be the *net area* of timber required. The *gross area*, or the actual sectional area of the timber, must be one and one-half times the net area, in order to allow for cutting away in framing, notching, and boring for bolt and rod holes. When the gross sectional area of a timber used as a tension member has been found, the least dimension that may conveniently be assumed is divided into this, and the other dimension thus determined. It is customary in all truss construction to make the principal timbers of uniform width, in order that the framing at the joints may be simplified, and the face of the truss may be flush.

EXAMPLE.—A tension member is required to withstand a stress of 13,500 pounds. If the undressed width of the principal rafter member is 4 inches, what will be the required size of the tension member?

SOLUTION.—The allowable tensile resistance of commercial spruce is about 1,000 lb. The gross section required is $13{,}500 \div 1{,}000 = 13.5$ sq. in. One and one-half times this result equals 20.25 sq. in. Dividing this result by 4, the width of the member that is fixed by

35—31

consideration of the design, the calculated dimension is slightly more than 5 in. Since no timber is sawed whose dimensions are 4 in. by 5 in., the commercial size, 4 in. by 6 in., is adopted. A timber of this size could be dressed on all sides and still have ample section to resist the stress. Ans.

It is quite common, especially in small trusses, to make the compression and tension members of the same size, and they then possess a great excess of strength.

36. Laminated Compression and Tension Members.—It is sometimes advantageous to build a truss of thin planks bolted or lagscrewed together. But no matter how well the keying, bolting, etc. may be done, the action of the compression members must always, in conservative design, be considered as that of separate posts, and in calculating, the least dimension must be taken as the thickness of the plank used. The thickness of the planks composing the built-up member must in no instance be less than one-sixtieth of the unsupported length of the compression member. In building up a frame of planks bolted and screwed together, it is considered poor practice to use less than three planks, and tension members when thus built up should be so proportioned that two of the members are strong enough to withstand the entire stress, allowing for cutting, as in the example just given. The compression and tension members must be firmly fastened together with bolts or lagscrews, and in some instances keyed in order that the pieces may act in unison. Bolts for securing the timber should not be more than 3 feet apart, while when keys are deemed necessary they may be spaced 4 or 5 feet on centers.

CONNECTIONS

37. Joints in General.—It is always more difficult to secure the required resistance at the connections of a wooden frame than it is in iron or steel construction, since wood varies so much in its ability to resist shear and compression parallel with and perpendicular to the grain. Hence, in designing, great care must be taken not to exceed these various bearing values.

38. That the strength of a truss is as largely dependent on the strength of the joints as on the strength of the tension and compression members, is easily understood when it is considered that no member or structure is any stronger than its weakest point. If the joints are not designed with care and accuracy, the concentration of stress at these points will greatly endanger the stability of the structure. Therefore, in proportioning wooden trusses, a force acting at an angle with the grain should be resolved into two components, one parallel with and the other perpendicular to the grain, and the areas of the cuts in each particular case should be proportioned to the area found by dividing these components by the safe bearing value of the wood.

39. Strut Joints.—Although these joints are comparatively simple they are none the less important and should not be neglected. In designing **strut joints** it is desirable that the struts, or ties, be so arranged that their center lines intersect the center lines of the rafter and tie-members. Sometimes this is impossible, and in such cases the loads resting on the joints may be placed so that they counteract the rotation and consequent bending moment. The direction and size of the cuts should be carefully considered, although in many cases these cuts are made without properly considering the stresses produced, or the strength of the timbers resisting the stresses.

When a strut bears against a compression or tension member at any angle but a right angle, its stress should be resolved into two components each parallel to one of the cuts of the joint offering the resistance. Each of these components should, in turn, be resolved into two components, one parallel with and the other perpendicular to the grain of the strut, and each is also resolved into its components parallel with and perpendicular to the grain of the rafter member, and the areas may then be calculated for the safe stress according to the intensities of these components. Cases arise in which it is impossible to obtain sufficient area to resist the stress, in which instances the use of a cast-iron

or wrought-iron shoe will reduce the unit pressure to a safe limit. When metal ties extend through the cord members, washers must be used to reduce the stresses to their allowable limit.

40. There are two general types of strut joints, those connecting the strut to the upper chord and those connecting it to the lower chord. Although they are the same in principle, they differ slightly in their design.

A detail of the simplest form of strut joint is given in Fig. 11. Here the center lines of the strut, rafter member,

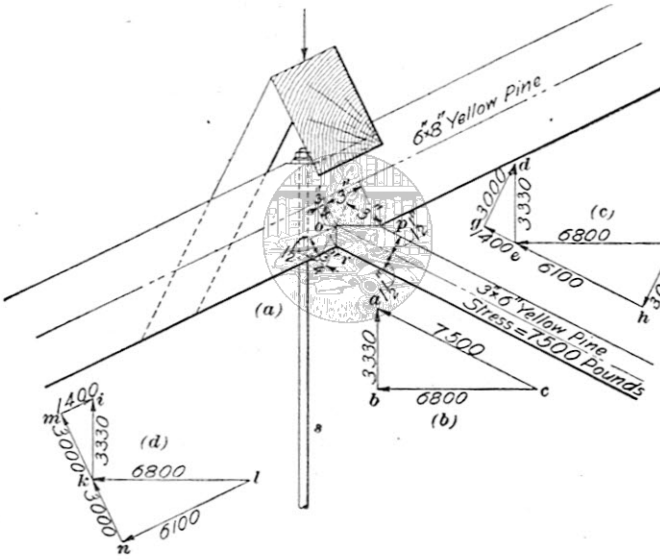


Fig. 11

and tie-rod intersect at a point corresponding with the panel point in the stress diagram. The joint is formed by notching the end of the strut into the compression member, the cut of the notch *op* being made to bisect the angle *p* between the strut and rafter member. This method of forming the cuts has the advantage of being convenient and usually disposes of the stresses to the best advantage. The principles and calculations involved in the design of this detail are illustrated in the following example:

EXAMPLE.—The stress in the strut shown in Fig. 11 is 7,500 pounds. Determine whether the cuts are large enough to reduce the stress per square inch so that the following allowable unit bearing values of the timber are not exceeded: Allowable unit compression on end grain, 2,000 pounds, and allowable unit compression perpendicular to the grain, 600 pounds.

SOLUTION.—In Fig. 11 (*b*), draw the line ac parallel to the strut, and lay off on this line, to some convenient scale, a stress of 7,500 lb. From the ends of this line draw ab and cb perpendicular to the cuts of the joints op and or, respectively. These components then represent the amount of stress that is taken by each of the cuts. In (*c*), to any scale, lay out these components from d to e and from e to f. Since the stress de represents the amount of compression on the cut op, by resolving this compression into its components parallel and perpendicular to the grain of the strut, it can be easily figured whether this surface is large enough. Draw eg parallel with the grain of the strut and dg perpendicular thereto. The component dg is found to be 3,000 lb. and is distributed across the projected area of the cut at the end of the strut, which, in this case, is 3 in. $\times$ 6 in. = 18 sq. in. The compression perpendicular to the grain is therefore $\frac{3000}{18}$, or 166 lb. per sq. in., which is much less than 600 lb., the allowable unit compression perpendicular to the grain.

The compression on this same cut parallel to the grain is 1,400 lb., as determined from ge in the diagram at (*c*), and the area resisting this is equal to the projected area of the cut on a plane at right angles to the stress from the strut and amounts to 6 in. $\times$ $1\frac{1}{2}$ in. = 9 sq. in. The compression per square inch on this surface is therefore $\frac{1400}{9}$, or 155 lb., which is well within the allowable limit of 2,000 lb.

The cut or of the strut in (*a*) has a direct stress of 6,800 lb. on it, and when this is resolved into its components parallel with and perpendicular to the grain of the strut, the stresses are found to be 6,100 lb. and 3,000 lb., respectively. The area resisting the compression perpendicular to the grain is $\frac{3}{4}$ in. $\times$ 6 in. = $4\frac{1}{2}$ sq. in., and the compression per square inch is $\frac{3,000}{4\frac{1}{2}}$, or 666 lb., which is slightly in excess of the allowable limit, though not dangerously so. The compression parallel to the grain is 6,100 lb., and the area resisting this is $1\frac{1}{2}$ in. $\times$ 6 in. = 9 sq. in. The compression per square inch is $\frac{6100}{9}$, or 677 lb., which is much less than the allowable bearing of the timber on the end wood.

In a similar manner the stresses on the rafter member are also found and the areas of the several cuts checked to determine whether the allowable bearing values are exceeded.

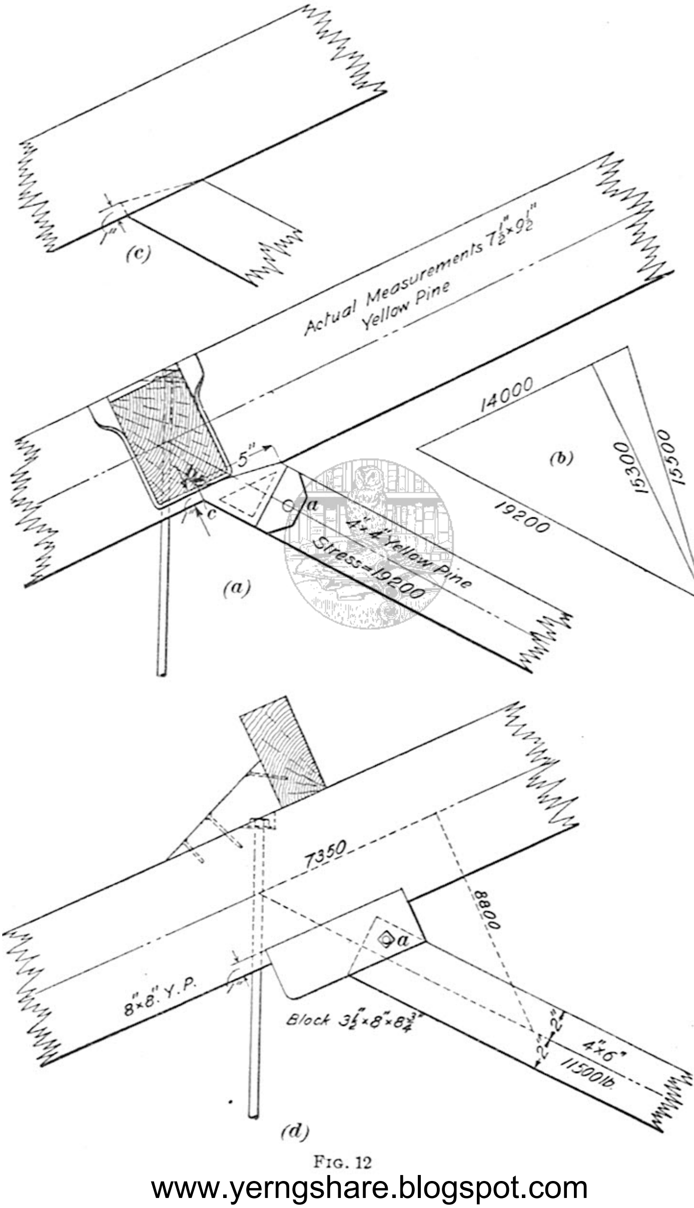
The diagram showing the intensity of these two components is shown in Fig. 11 (*d*) and is composed of forces of the same amounts as those in the diagram (*c*). It is evident that the projected areas of

the cuts in the rafter member are the same as those for the end of the strut, and the stresses to be resisted are likewise of the same amount, that is, the compression on the joint *o r* parallel to the grain is 6,100 lb., and its area is 9 in., which is the same as already figured for the strut. Hence, the joint is amply safe from failure by crushing of the timbers in any direction, except that the stress for the projected area perpendicular to the grain of the joints is slightly greater than the allowable unit stress given in the example, but this excess is so slight that it may be disregarded in practice. Ans.

In the connection just analyzed, the tie-member *s* sustains very little stress, its function being to support the lower chord of the truss. Hence, it is not customary to calculate its size, but to make it of either a $\frac{3}{4}$-inch or $\frac{7}{8}$-inch round iron bar. In a truss no rods of less than $\frac{3}{4}$ inch diameter should be used. The ends of smaller rods are likely to twist or break off if the nuts are tightened too much when drawing the joints together.

41. A detail of a strut joint that may be used to advantage where it is required to increase the bearing area of the strut is given in Fig. 12. This design is also useful when it is desired to cut the compression member as little as possible. In this case it is assumed that the cut in the rafter member may not be deeper than 1 inch. If the cut is made as shown in the figure, the compression perpendicular to the face *b c* is found, by the method in (*b*), to be 14,000 pounds. This force will require an area of $\frac{14000}{2000}$, or 7 square inches. If the $4'' \times 4''$ strut had been framed into the rafter but 1 inch, as required by the conditions of the problem, and as shown at (*c*), the area would have been too small. There is therefore introduced a cast-iron cap that extends the cut the entire width of the rafter member, giving an area of 7.5 square inches, which is sufficient.

The direct compression on the rafter member perpendicular to the grain is 15,300 pounds, and the allowable compression is $5 \times 7.5 \times 600 = 22,500$ pounds. This area is large enough to resist the compression perpendicular to the grain of the rafter member. The bolt shown at *a* secures the cap and the strut member together.



(c)
Actual Measurements $7\frac{1}{2}" \times 9\frac{1}{2}"$
Yellow Pine
14000
(b)
15300
15500
19200
5"
a
c
4"×4" Yellow Pine
Stress=19200
(a)
7350
8800
a
8"×8" Y.P.
Block $3\frac{1}{2}" \times 8" \times 8\frac{3}{4}"$
2"
2"
4"×6"
11500 lb.
(d)

FIG. 12

When stresses are light, no shoe is required, and the strut is simply notched into the rafter member, as in Fig. 12 (*c*), while in cases where a shoe is unavailable the strut may be framed into a white-oak block let into the rafter, as shown in Fig. 12 (*d*).

42. Fig. 13 illustrates a joint in which the stresses to be resisted are so great that a cast-iron shoe is required for the strut bearing. It is useful where the angle between the strut and compression member is small. The washer

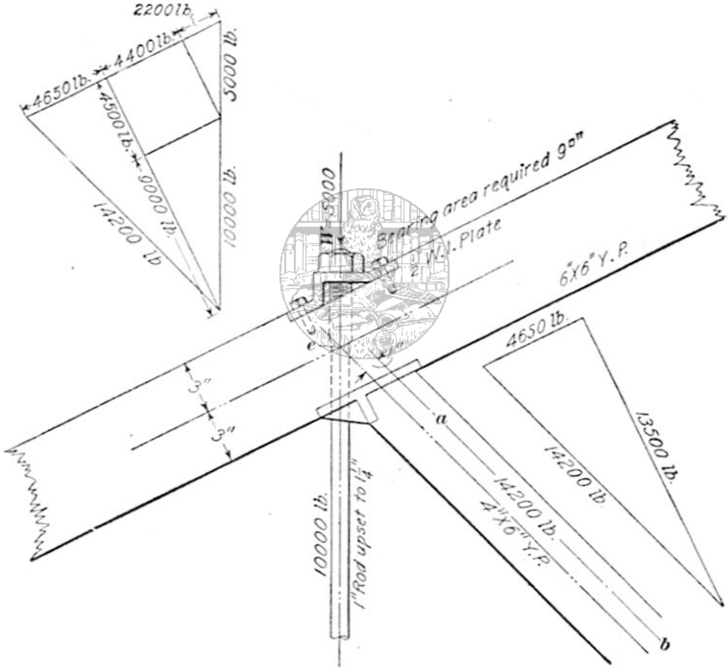


Fig. 13

shown is adapted to cases where, on account of the angle between the strut member and the rafter member, the stress parallel with the grain of the rafter member is reduced, and where it is desired to cut the compression member as little as possible. It is well to use lagscrews to hold the casting in place, but in many cases this precaution is not observed.

43. The strut joint given in Fig. 14 (*a*) is of advantage where the stresses are so great that in order to resist them, the end bearing of the strut must be increased. This is accomplished by introducing a cast-iron bearing plate that distributes the pressure of the strut over a large area of the upper chord member. This plate is provided with flanges cast on the sides between which the end of the strut member enters, and which serve as a guide and help to prevent this member from being displaced.

In this case very little cutting is necessary, as the strut or compression member is always perpendicular to the upper

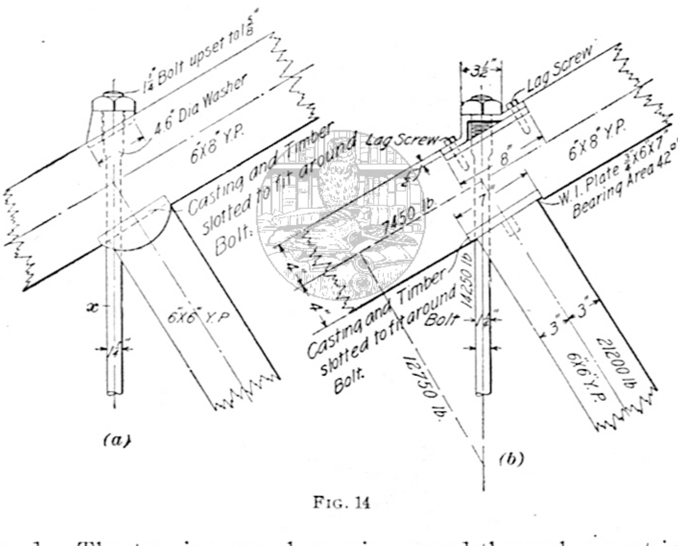


FIG. 14

chord. The tension member x is passed through a cast-iron washer of a design that is very useful where a number of trusses are to be built, as such washers are used throughout the upper chord. In such instances, it is advisable to proportion the bearing area of the washer for the tension member sustaining the greatest stress. The excess of area thus provided for the tension members of less stress is not objectionable, since but one pattern is required and complications resulting from the change of size are avoided. If the tension rods vary in diameter, the holes in the washers must be cored to suit.

In Fig. 14 (*b*) is given a method of forming a secure connection in which a wrought-iron plate is used instead of a casting, while in place of the side flange, a pin is employed to guide and hold the strut in position. Provision is made to resist the great stress by substituting a built-up steel-plate washer to supply the necessary bearing on the upper surface of the rafter member. It is not necessary to rivet the strap to the bent piece, for the lagscrew provides sufficient security against the tendency of the bent piece to spread. The holes through the straps for these lagscrews should be of such a size that the screws will fit them securely.

The detail shown in (*b*) may be used where large stresses are encountered. In many cases, especially where but few trusses are to be built, it is found cheaper and more convenient to have these washers made up than to use the cast-iron washers, as the saving on the cost of the patterns compensates for the cost of labor on the wrought-iron washers.

44. Another variety of cast-iron cap used for the connection of the strut to the compression member is shown in Fig. 15. Brackets are provided on each side to supply support for the purlins. The required resistance against sliding is obtained by setting the casting in the rafter member to the depth of the two projecting pieces *a*, *a*.

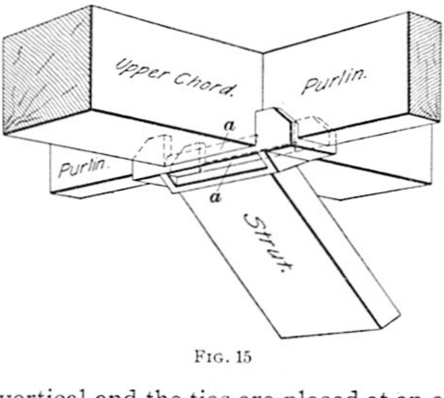


FIG. 15

45. All the details that have been shown were for trusses in which the struts slant and the ties are vertical. In the design given in Fig. 16 the struts are vertical and the ties are placed at an angle. The sliding component due to the stress in the tension member is taken up by the cast-iron washer, which is provided with the lip *a b* at the upper end. In designing this detail, care should be

taken that the area of the cut *a b* is large enough to reduce the unit end compression on the rafter member to the safe stress, and that the shear along the line *a c* does not exceed the allowable shearing stress of the timber. The bending moment on the metal at the point *b* should not be so great as to exceed the allowable unit transverse stress of cast iron, which is assumed to be 5,000 pounds. In this detail the vertical compression member is much smaller than the upper chord and instead of using any block or casting to connect it with the compression member, the strut has simply been let into the rafter member its full width.

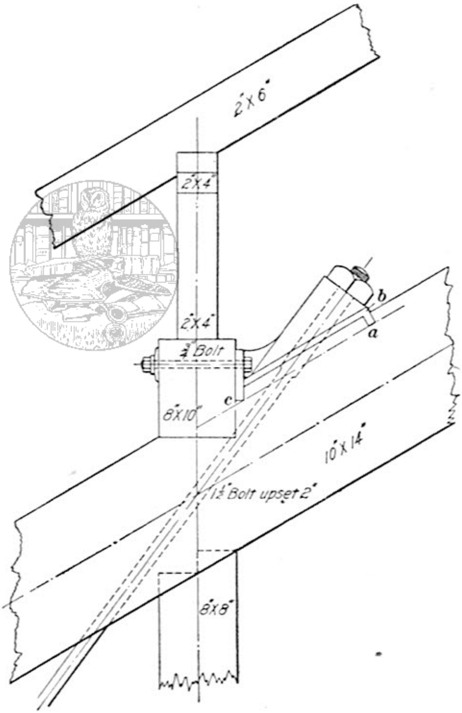


FIG. 16

46. Fig. 17 illustrates the connection between the lower end of a strut and

be balanced by placing the purlin so as to produce an equal and opposite moment around the joint. The principles involved in the design of connections at which the struts are misplaced are illustrated by the following example:

EXAMPLE.—If the center line of the 4″ × 6″ yellow pine strut shown in Fig. 13 had been so placed as to coincide with the dot-and-dash line, at what distance from its present position would it be necessary to place the load, in order to balance the bending moment produced?

SOLUTION.—Since this strut is 1 in. from the correct line, the moment produced is 14,200 × 1 = 14,200 in.-lb. This is to be balanced by a load of 5,000 lb. placed on the truss at a distance from the point *e* equal to $\frac{14200}{5000}$, or 2.84 in., which is the distance the load must be moved horizontally toward the right.

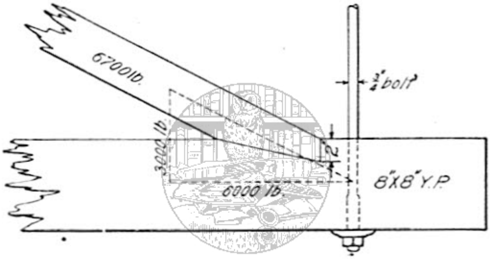


FIG. 17

48. Peak Joints.—A few of the many varieties of **peak joints** are given in Fig. 18, the simplest of which is detailed in (*a*), where the tension rod passes between the two compression members and is held by a wrought-iron washer. For light construction this joint is very convenient and economical. The area under the washer should be sufficient to resist the tension in the rod both for end and side compression on the wood, and the thrust of the two compression members should have sufficient area along the face *a b* to avoid exceeding the allowable compression of the timber per square inch.

In (*b*), two wrought-iron plates are fastened to the sides of the compression members, and the washer under the nut of the tension rod is made wide enough to rest on these plates, thereby relieving the compression member of the direct stress from the tension rod. In Fig. 18 (*c*), a

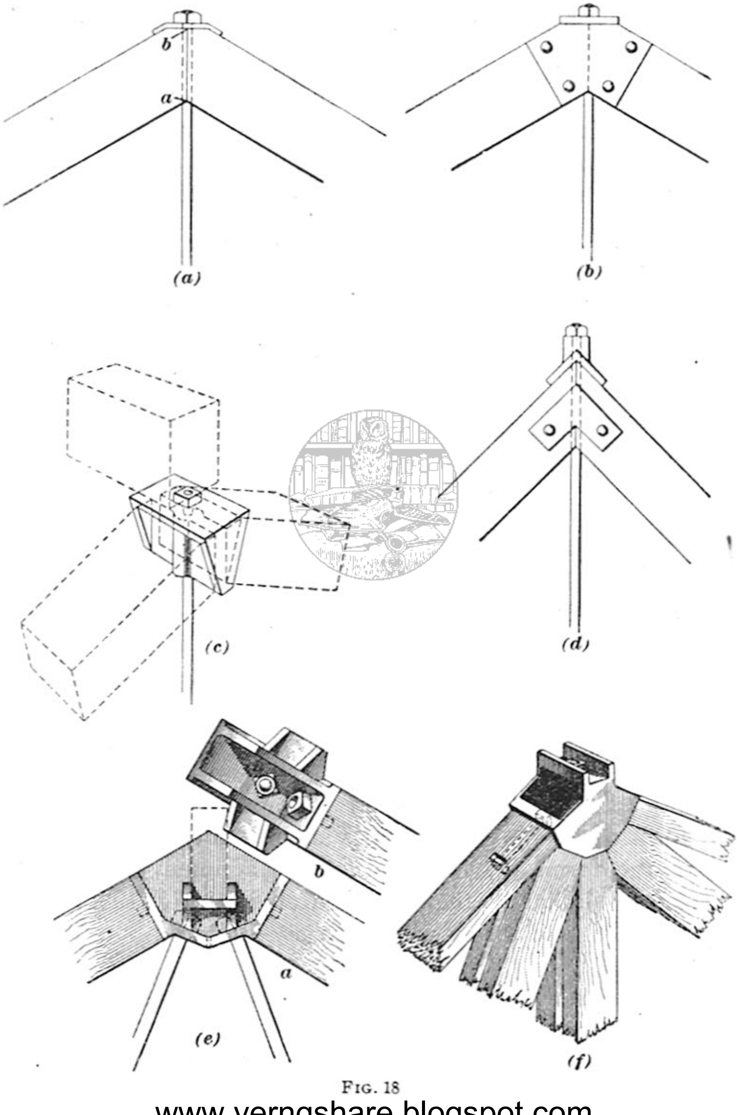


FIG. 18

cast-iron cap is introduced; this makes a very complete and efficient joint and is applicable to trusses in which the stresses are greater than would exist in (*a*) and (*b*).

The application of a cast-iron angle washer to a roof truss of steep pitch can be seen in (*d*), in which case the washer is made to fit the peak of the truss. If two tension members must be accommodated at this joint, a detail similar to the one in (*e*) may be adopted. On either side brackets are cast, into which the ridge pole is slipped, and a small hole is bored in the bottom of the casting to permit the escape of any water that might accidentally collect there.

The type in Fig. 18 (*f*) may be employed with advantage in cases where the stresses in the several members meeting at the peak are subjected to frequent changes by the wind shifting so as to affect first one side and then the other. The members coming together at the peaks are tension rods, or bolts, and wood compression pieces. The sides are left open to permit the tightening of the bolts throughout the connection. The top of the casting is raised and finished in the form of a bracket to form a support for the ridge pole. When the forms (*a*), (*b*), and (*d*) are adopted, a wrought-iron strap similar to the one illustrated in Fig. 19 may be employed to hold the ridge pole firmly in place.

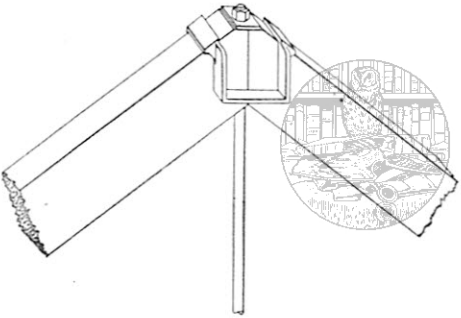
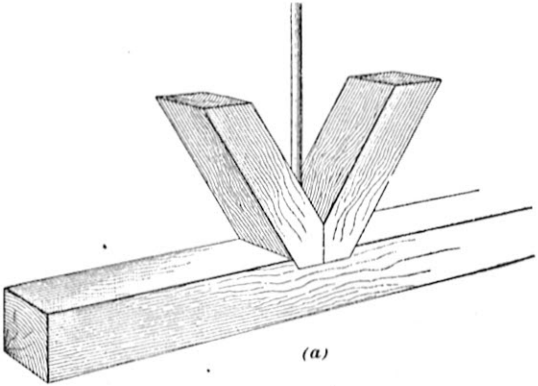
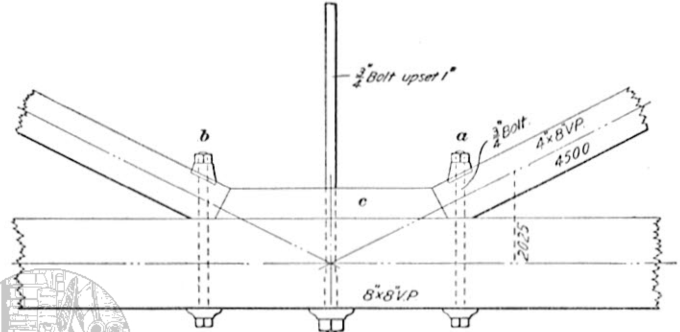


FIG. 19

49. Center Joints.—In Fig. 20 several types of **center joints** for wooden trusses are illustrated. In (*a*), the two struts butt against one another and are let into the tension member for a short distance, as a precaution against displacement. In (*b*), the angle formed by the struts and the tie-member is so small that a hardwood block *c* is inserted

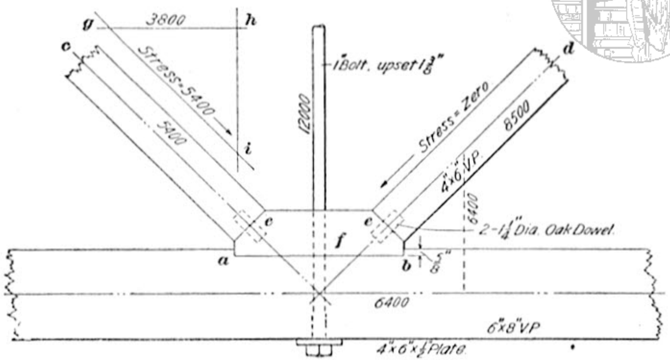


(a)

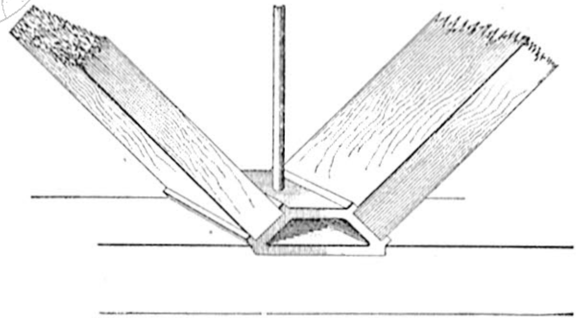


b
a
3/4" Bolt
4"x8" V.P.
4500
c
2025
8"x8" V.P.

(b)



g
3800
h
c
Stress=5400
5400
i
1" Bolt, upset 1 3/8"
12000
d
Stress = Zero
8500
4"x6" V.P.
6400
e
e
f
2-1 1/4" Dia. Oak Dowel.
a
b
5/8"
6400
6"x8" V.P.
4"x6"x1/2" Plate.



(d)

between the ends of the struts to hold them in position without cutting the lower chord or tension member, and bolts are employed at *a* and *b* to make the joint more rigid.

50. In all trusses subjected to any eccentric load, as a wind load, the stresses are so variable that at a center joint where two struts meet and oppose each other, one must often withstand a greater stress than the other. When this variation is but slight, it is provided for by bolts *a*, *b*, Fig. 20 (*b*), but when considerable difference exists the joint or connection should be detailed as in Fig. 20 (*c*). Here the struts are maintained in position by oak dowels *e*, *e*, and the block, or bolster, on which they bear is let into the tie-member.

In order to figure to what depth the tension member must be cut to receive the block *f*, the difference in stress on the two struts should be determined from the wind-load diagram. The block is held in place by the thrust of the struts and the tie-member, although it is very important that the cuts at *a*, *b* be made with the greatest care in order to secure a tight fit, which may, in many cases, be more readily accomplished by making these cuts with a slight taper. The necessity of a tight fit between the block and the tie-member is obvious, as any change of stress would cause a slight play in the block, causing it to rock back and forth, which would be detrimental to the stiffness of the truss.

The method of figuring the area of the cut for the block is illustrated by the following example:

EXAMPLE.—In Fig. 20 (*c*), the stress due to the wind load in the member *c* is 5,400 pounds when the stress from the wind is zero in *d*; will the cut shown in the figure supply sufficient bearing area at *b*, providing the safe unit stress of the timber in end bearing is 1,600 pounds?

SOLUTION.—Draw the triangle of forces at *g h i*, resolving the stress of 5,400 lb. into its horizontal and vertical components. From this it is found that there is an unbalanced horizontal force of 3,800 lb. tending to shift the block to the right. The area of the cut must therefore be equal to $\frac{3800}{1600}$, or 2.375 sq. in. Since the width of the timber is 6 in., the required depth is $\frac{2.37}{6}$, or about .4 in. The actual depth shown in the figure is $\frac{5}{8}$, or .625 in., so that the distance the block is let into the tie-member will be ample.

In the type shown in Fig. 20 (*d*), a cast-iron block is used, and the struts are held in place by projecting lips made on the casting.

51. Heel Joints for Light Loads.—More difficulty is experienced in designing the **heel joint** than any other, for the reason that in a truss the stresses accumulate until the heel is reached, and are greatest at that point. On this account the heel is the most important joint, and, in timber trusses, usually requires careful study in order to secure the necessary resistance.

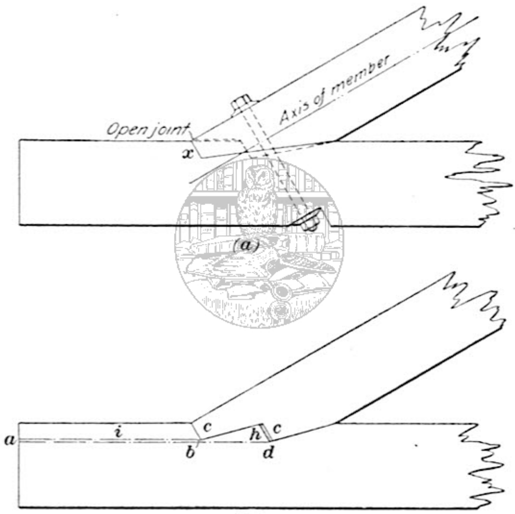


FIG. 21

The simplest type of heel joint, shown in Fig. 21 (*a*), is adapted to trusses in which the stresses are small. The portion of the tension members projecting beyond the notch has sufficient shearing resistance to take all the horizontal pull in the lower chord, while a bolt holds the members in position. An objection might be raised that the area *x*, which resists the thrust of the rafter, is not concentric with the line of stress, which is considered as coinciding with the axis of the member. A suggested improvement is

indicated by the dotted lines, in which the resisting area is made concentric with the thrust of the upper chord. In order that the thrust of the rafter may not have a tendency to break off the toe of the member, the end is left with an open joint, as shown.

It is seldom advisable to make two cuts, as in (*b*), because through careless workmanship and by shrinkage, one of the bearings at *c*, *c* is sure to become inactive, leaving the other to withstand all the stress, so that the shear along either *a d* or *a b* only will be realized and the pieces *h* and *i* will shear off successively instead of conjunctively, greatly weakening the joint.

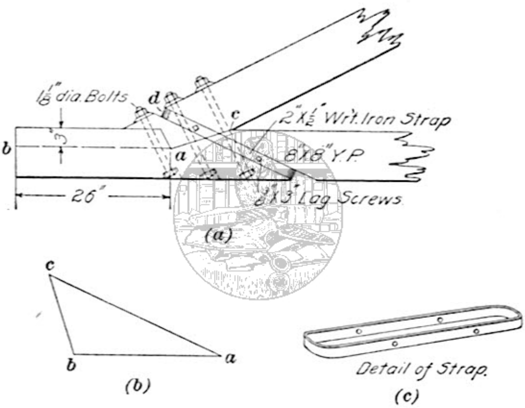


FIG. 22

52. When the shearing strength of the projecting member is not sufficient to resist the thrust of the upper chord, the design shown in Fig. 22 may be adopted. The extra strength required is supplied by the use of a wrought-iron strap, the bolts serving to make a more rigid connection, although they are not considered when figuring the resistance of the joint. Such a joint should be used only when well-seasoned timber can be secured. The calculations for determining the strength of the joint are given in the following example:

EXAMPLE.—If, in a joint constructed as in Fig. 22, the stress in the rafter member is 39,000 pounds and in the tension member, 37,300 pounds, what size wrought-iron strap will be required, providing

the dimensions of the timber and the connections are the same as those given in the figure, and the allowable unit shearing resistance of the timber parallel with the grain is considered as 120 pounds, and the allowable unit tensile stress of the strap is taken as 16,000 pounds?

SOLUTION.—Since the length from *a* to *b* is 26 in. and the width of the piece is 8 in., the area resisting the thrust is 26 in. by 8 in. = 208 sq. in., which multiplied by 120 lb. gives $208 \times 120 = 24,960$ lb., shearing strength. Then the remaining stress to be taken care of by the strap is 37,300 lb. − 24,960 lb. = 12,340 lb.; for convenience, say 12,400 lb. In the diagram shown in (*b*), lay off, to some convenient scale, *a b* equal to 12,400 lb. This will be one component of the stress in the strap. The other component exists as a pressure on the cut *a c* of the connection, as shown in (*a*). In the diagram at (*b*) draw *a c* parallel with the center line of the strap shown in (*a*), and *b c* perpendicular to *a c* in (*a*); this locates the point *c*. Then, by the same scale, measure the length of the line *a c*. This length represents the stress that the strap will be required to resist, and is found to equal 15,750 lb. As the strap is 2 in. by $\frac{1}{2}$ in., the area on both sides is 2 sq. in., so that if a safe unit fiber stress of 15,000 lb. is assumed, the strap will have a tensile resistance of 30,000 lb. Thus, it is observed that the strap has a resistance nearly double that actually required; although, on account of the fact that if the strap were made narrower, it would, under the full stress, be likely to crush or cut the timber at *d*, Fig. 22 (*a*), its width cannot well be reduced. Besides, the strap must be welded in at least one place, and it must be drilled for the lagscrew, which will reduce its gross sectional area to about 1.563. These facts, when considered, lead to the conclusion that the strap as designed is none too large, for it is not safe to figure on more than 75 per cent. of the strength of the section where a weld occurs. Ans.

53. In Fig. 23 is given a method for framing the heel joint of a truss with a steep pitch. A cast-iron shoe with a lip *a* is employed. The strength of the joint depends on the strength of the lip and the shearing strength of the tension member *b* along the line *c d*. In designing this connection no reliance is placed on the bolts and lagscrews.

The figure also illustrates a possible construction for framing the back wall and roof. Here a $3'' \times 12''$ wall plate is bolted to the end of the tie-member *b*, and supports the rafter along its length, the brickwork being carried up on each side of the truss to the under side of the plate. The ends of the rafters are covered and protected by a galvanized-iron cornice *e*.

54. In Fig. 24 is shown an excellent detail adaptable to trusses of intermediate spans, or spans of from 30 to 50 feet. The special advantage of joints of this type is that a strap that can be so adjusted as to be drawn tight and thus take up any shrinkage that may occur in the timber is used. The design has the further advantage that no bolt holes need be bored in the timber, thereby effecting a saving, and avoiding the necessity of cutting the members.

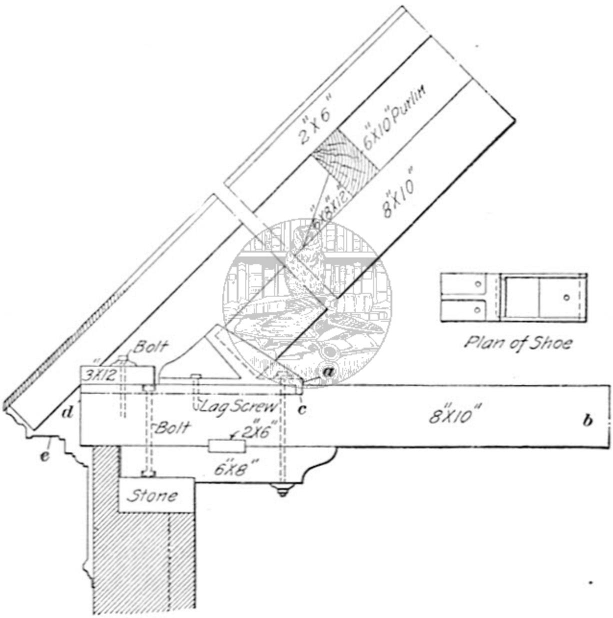


Fig. 23

The tendency of the strap to crush the corner of the timber around which it passes is overcome by placing a casting *a* on top of the upper chord, while a casting *e* shaped so as to form a square bearing for the nuts is used at the lower end. The details of construction are clearly designated in that portion of the illustration showing a section on the line *x x*. It will be noticed that the lower chord of the truss projects far enough beyond the foot of the rafter member to give sufficient area to resist the whole thrust, and for this reason a rod

$\frac{3}{4}$ inch in diameter is sufficient to hold the joint in position. But if the rod is proportioned to stand the whole stress, it must have a diameter of $1\frac{1}{4}$ inches at its weakest section, which occurs at the root of the thread. The calculations for the strength of this strap are similar to those given for Fig. 22.

55. Laminated Truss Heel Joints.—The constructions shown in Fig. 25 are usually adopted when the tension and compression members are made of plank. Although some authorities claim that bolts should never be used as pins, since the length is so great in proportion to the

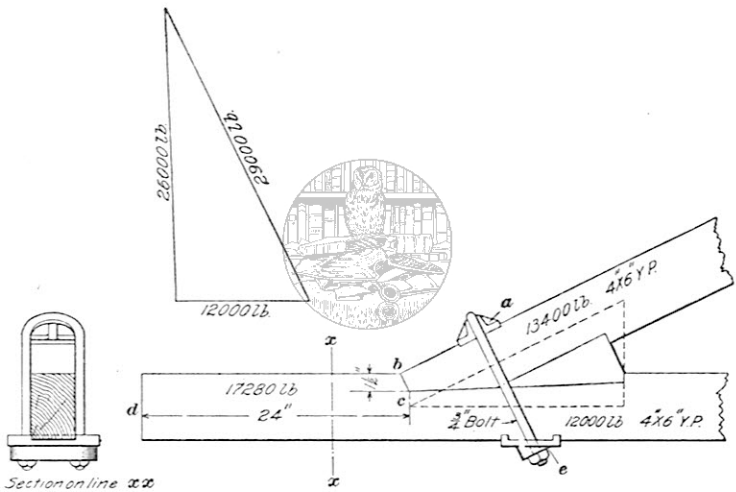


Fig. 24

diameter that the bending moment is excessive, they may be employed with advantage in cases where the stresses are not very great. In the present case the bolts have an effective span of 5 inches, as shown in the end view, and if the joint is designed as in (a), the bolts must be proportioned to stand the full stress of the compression members.

If the bolts are considered as resisting equal shares of the stress from the oblique or compression members, each is assumed to support a concentrated load of $\frac{2,225 + 2,225}{4}$ $= 1,112$ pounds. From this load each bolt will be subjected

to a bending moment of 1,390 inch-pounds, which is calculated by applying the formula

$$M = \frac{WL}{4} \qquad (4)$$

in which W = concentrated load of 1,112 pounds;
L = distance between centers of supports, or 5 inches.

To resist this bending moment there will be required a section modulus equal to $\frac{1390}{12000}$, or .116 for each bolt, provided

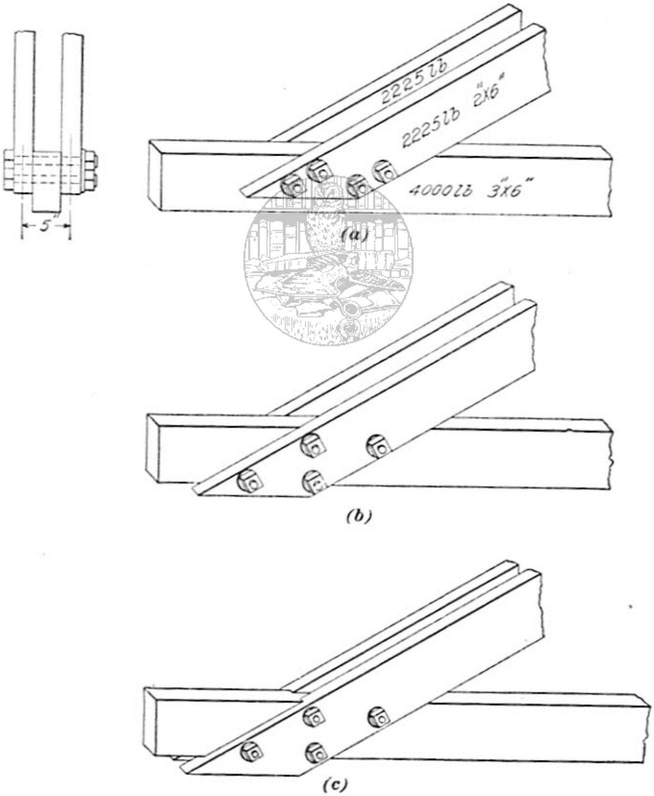


FIG. 25

vided that the safe unit fiber stress is taken at 12,000 pounds. Calculated according to the formula, $S = .0982\ d^3$, the section

modulus of a 1-inch bolt is .0982 and that of a $1\frac{1}{8}$-inch bolt is about .139. The former, though somewhat light, can be used with safety.

If the compression member is carried down, as shown in Fig. 25 (*b*), so that it rests directly on the bearing block, the vertical component of this stress is balanced, and the only stress to be resisted by the bolts is the direct pull in the tension member. In this instance the required section modulus for one bolt is only .104, which shows a saving over the former construction, or at least a greater margin of safety.

If the tension member had been let into the compression member a short distance, as in (*c*), the bending moment on

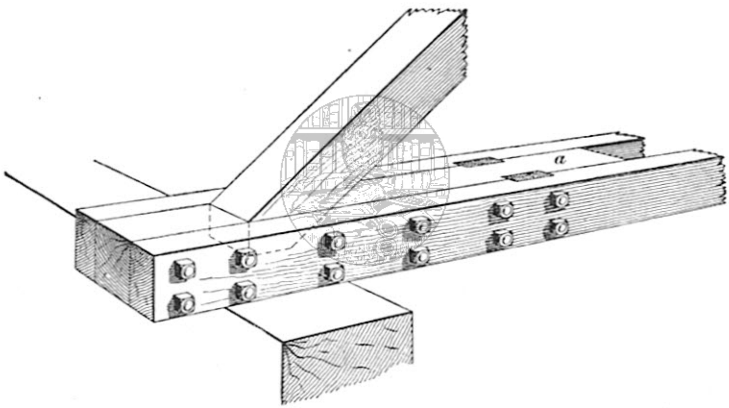


FIG. 26

the bolt would have been entirely eliminated and the only office performed by the bolts would be to hold the parts together.

In order to secure alinement of the holes they should be bored when the pieces are in position, the work being done very carefully in order to secure a snug fit for the four $\frac{3}{4}$-inch bolts that are required for a joint of this design.

56. Fig. 26 illustrates a detail applicable to trusses made up of small members. To avoid cutting the tension pieces from the top surface, a block *a* is inserted between the sides of the two tie-members and secured to them by keys and

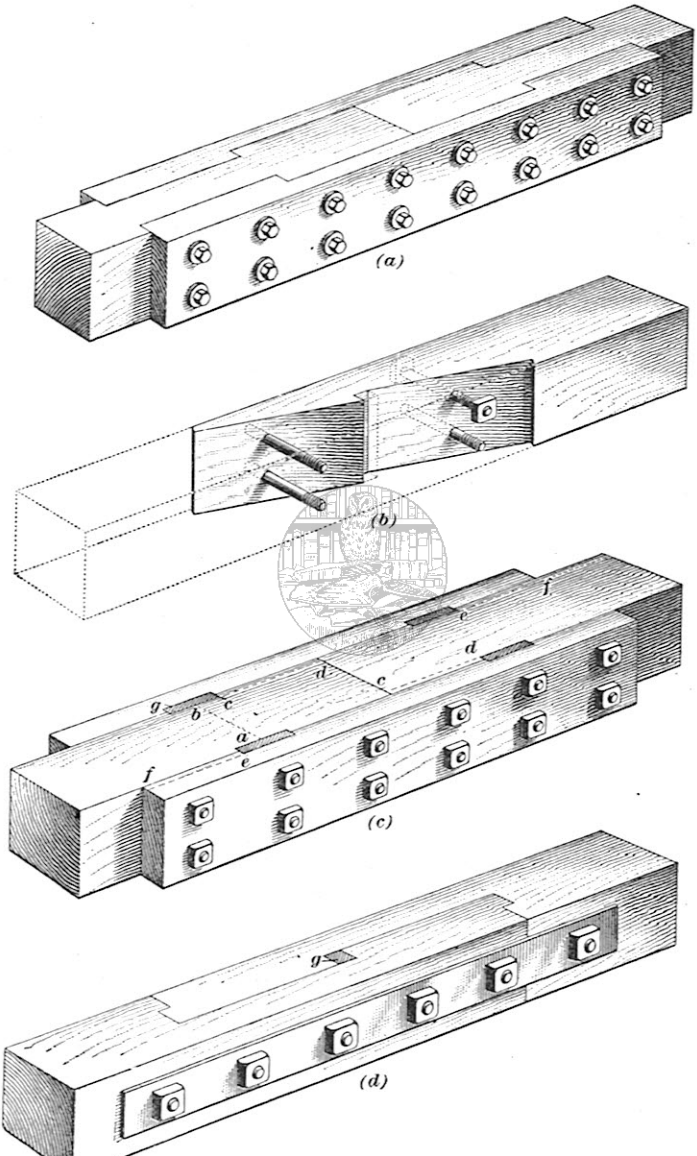


FIG. 27

bolts. The compression member is let into this block to a sufficient depth to avoid any possibility of inadequate bearing, while its load or stress is depended on to prevent its being displaced.

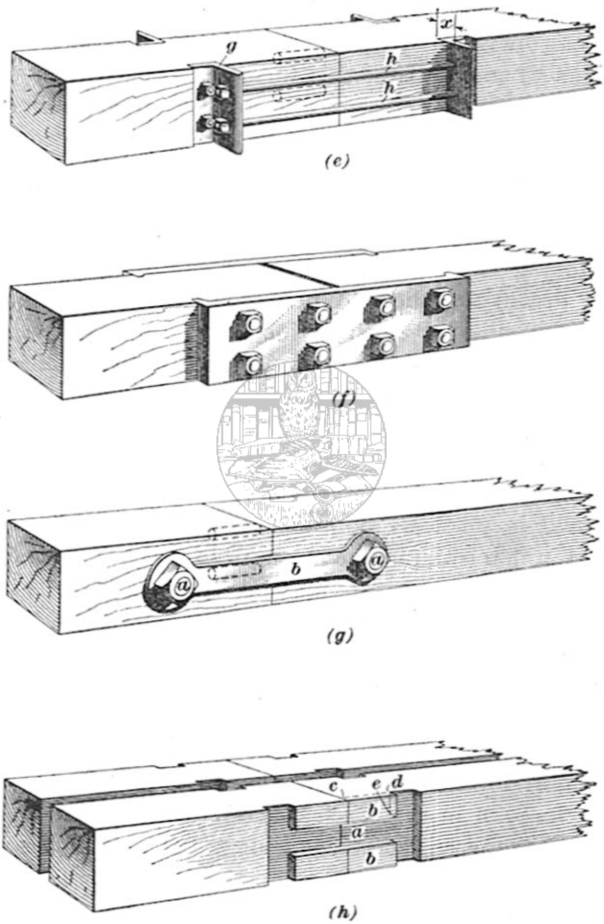


Fig. 27

57. Splices.—A number of splice joints for wooden tension members are illustrated in Fig. 27, the one in (*a*) being formed by butting two pieces together and using two splice plates of wood that are let into two opposite sides of

the tension members, and held in position by means of bolts. The splice in (*b*) is varied by being cut obliquely and toothed, and is tightened by the use of keys and held together by bolts. This is very convenient in some cases, but is not as strong as the type in (*c*). In (*d*), the cut is made lengthwise of the piece, and the joint is strengthened by means of metal bars; this form may be used in trusses that have a great deal of stress coming on the connection. In the splices in (*e*), (*f*), and (*g*), metal is employed to supply the required strength and pins are used to guide the members and keep them in line. In (*f*), the ends of the cast-iron plates are made with a slight bevel, so that in tightening the bolts the two parts of the tension member are drawn together. When the truss under consideration is of laminated construction, such a detail as the one shown in (*h*) forms a simple and effective splice; this design is especially useful where it is desired that the full width of the splice shall not exceed the width of the tension piece. A wrought-iron plate is cut to the form shown at *a* and set into the wood. One such plate is used on each side, being held in place by bolts or lagscrews. The principles involved in determining the strength of a splice joint or connection of a tension or tie-member are the same as those explained in connection with the detail design of heel and strut connections.

In the design of all the splice connections shown in Fig. 27, after the maximum stress in the member has been obtained from the stress diagram, it is necessary to investigate the joint usually for failure in three ways. For instance, referring to the joint in Fig. 27 (*c*), it is first necessary to determine whether there is sufficient sectional area of timber on the line *a b* to resist the stress in the tie-member; then it must be decided whether the timber used for the tie-member supplies sufficient resistance to shearing along the line *c d*, and third, the wood splice plates along the line *e f* must be considered in order that it may be known whether there is sufficient resistance to supply the necessary stress. It is also well to investigate the bearing on the end wood of such

surfaces as those marked *g*, *g*, Fig. 27 (*c*), (*d*), and (*e*); also, in cases where iron is used, as in (*e*), the bolts *h*, *h* in tension must be proportioned to withstand the entire stress to which the member is subjected, and in determining the strength, the area at the root of the thread is to be taken, unless the bolts are *upset*, or enlarged on the ends so that the area at the root of the thread will be at least 10 per cent. greater than the area of the body of the bolt.

The angles are subjected to bending from the pull of the bolts, the bending moment being equal to the tension in the bolts *h*, *h* multiplied by the distance *x*. The resistance that the angles offer to this bending stress should be calculated, and if the resistance is less than required, angles of greater weight must be used.

The detail in Fig. 27 (*g*) is weak from the fact that the bolts *a*, *a* bear directly against the wood, and are apt to crush or split the wood before their full strength can be realized. A better arrangement is provided by setting the bar *b* into the timber, driving it tightly in place to make it fit well. As a rule, the resistance of the bolts in splice connections should be neglected, except in such constructions as in (*g*). In this case the bolt, which partakes more of the nature of a pin, should be of considerable size and analyzed for bending.

The principal points for consideration in the detail in Fig. 27 (*h*) are whether the pieces *b*, *b* have sufficient shearing resistance along the line *c d* and if there is enough bearing provided at *e d* on the end wood to realize a resistance equal to the stress.

58. An example illustrating the calculations necessary in determining the strength of a splice joint is given in the following:

EXAMPLE.—In Fig. 28 is shown the splice of a tie-beam in a wooden roof truss composed of yellow pine. What is the strength of the splice, disregarding the bolts *a*, *a* entirely?

SOLUTION.—The strength of the splice depends on the tensile strength of the wood at the net section *e f* and on the tensile strength of the net section of the two splice plates. It also depends on the tendency of the splice plates to shear along the lines *s t* and *s' t'*, and on

the tendency of the tie to shear along the lines $m\,n$ and $m'\,n'$. Assume the areas of the net sections to be sufficient to make their strength greater than that of the sections that will fail by shearing; then, referring to Fig. 28, it will be seen that the line of shear on the splice plate at $s\,t$ and $s'\,t'$ is longer than that of the tie-member at $m\,n$ and $m'\,n'$; therefore, in computing the strength of the splice, the strength of the

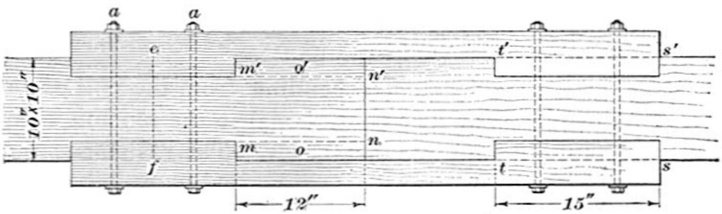


FIG. 28

tie need be considered along the lines $m\,n$ and $m'\,n'$ only. The pieces o, o' tend to slide or shear off from the main tie along the lines $m\,n$ and $m'\,n'$. The area along these lines is $12 \times 10 \times 2 = 240$ sq. in.; and since the ultimate shear of yellow pine parallel to the grain is 400 lb. per sq. in., the ultimate strength of the splice, disregarding the bolts a, a, is $240 \times 400 = 96{,}000$ lb. Ans.

59. Heel Joints in Trusses of Long Spans.—The detail given in Fig. 29 may be employed in cases where the stresses at the heel joint are great, and the projection of the tension member beyond the support or bearing of the rafter member is limited. To reenforce the tension member where the lip of the shoe cuts into it, a $6'' \times 6''$ bolster is introduced and is connected to the tension member by means of the $\frac{3}{4}$-inch bolts shown at a, a. The tendency of the tie-member or tension member to slide along the bolster b is resisted by keys k, k let into the bolster and tension member. The wall bearing is located directly under the junction of the center lines of the several members so that the tie-member is subjected to no bending stress, as it would be if this intersection were located outside the edge of the wall. The $\frac{7}{8}$-inch bolt g overcomes any tendency of the cast-iron shoe c to slip out of the cut provided for its lip d.

60. The type of heel connection illustrated in Fig. 30 may be effectively used in timber trusses of large span.

The bolts introduced in this detail greatly increase the strength of the joint. The calculations necessary for the design are as follows:

The outward thrust of the compression member, which is equal to the pull of the tension member, or 32,250 pounds, is resisted by the shear of the lower chord along the plane *b c*, and by the horizontal component of the tension in the bolts *d*, *d*. The allowable, or safe, shear of the wood along the plane *b c* is first calculated, and the bolts then propor-

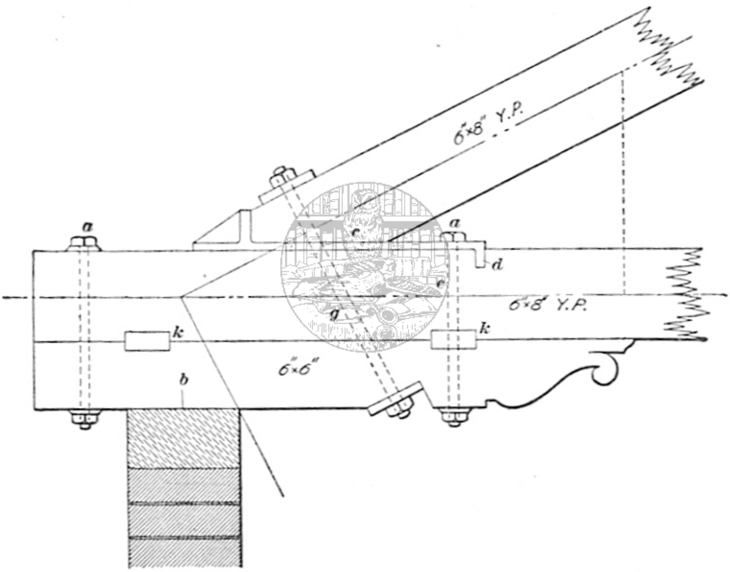


Fig. 29

tioned to resist the remaining stress by their horizontal components. The shearing strength of the wood along the line *b c*, when 120 pounds is taken as the allowable unit stress, is $120 \times 6 \times 29\frac{1}{2} = 21,240$ pounds, leaving a remaining stress of 11,010 pounds to be sustained by the bolts. Since the bolts are placed perpendicular to the upper edge of the rafter member, and the horizontal component of their stress is known, the triangle of forces $c'\,a'\,b$ may be drawn by laying off $c'\,d'$ in a horizontal direction equal to 11,010

pounds, and drawing $d'\,b'$ vertically, and $c'\,b'$ parallel in direction with the center line of the bolts. The line $c'\,b'$, which represents the amount of stress to be resisted by the bolts, scales 24,800 pounds. If it is considered that the bolts are of first-class rolled bar iron and have a safe unit tensile resistance of 18,000 pounds, then the combined net area of the bolts will need to equal $\frac{24800}{18000}$, or 1.37 square inches. The area of the root of the thread of a $1\frac{1}{8}$-inch bolt is .694 square inch, from which it is evident that as two such bolts will have a combined area at the root of the thread of 1.388 square inches, they will be sufficient. In order to

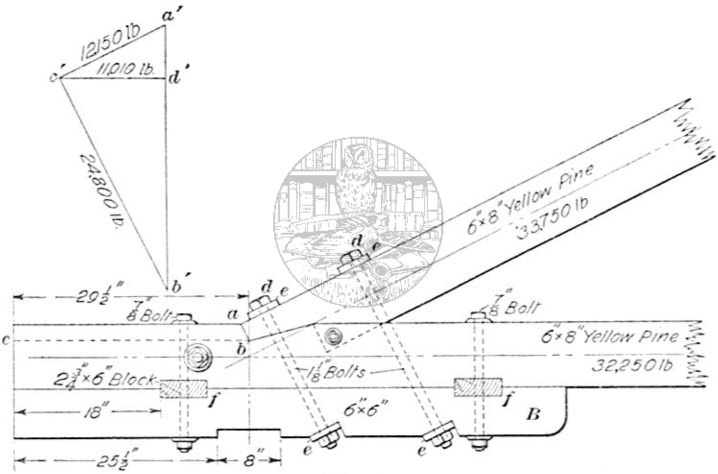


FIG. 30

obtain enough bearing area, so that the allowable unit bearing value of the timber perpendicular to the grain is not exceeded, the washers at e, e should extend across the entire width of the members. Georgia yellow pine, of which this truss is assumed to be constructed, has an allowable bearing, perpendicular to the grain, of about 600 pounds, so that if the pressure is 24,800 pounds, and the length of the bearing, or width of the timber, is 6 inches, the width of the washer will be $\frac{24,800}{600 \times 6} = 6.89$ inches. Hence, a washer made of a piece of 6-inch bar iron 7 inches long and having two $1\frac{1}{4}$-inch

holes bored in it would be practical. Such washers are usually made $\frac{3}{4}$ inch to 1 inch in thickness. In this case, however, two single meshes are used.

61. The bolts in this construction are carried through the tension member and the bolster B, which is used to avoid cutting the tension member for the bolts d, d. The horizontal component of the stress in the bolts is transmitted from the bolster to the tension members by means of blocks f, f. This horizontal component is 11,010 pounds; hence, if one of the blocks is located 18 inches from the end of lower chord member, the shearing strength of the wood parallel with the grain will be $18 \times 6 \times 120 = 12{,}960$ pounds, which will be sufficient.

The depth, or thickness, of the oak blocks is determined by the bearing area required at the edge of the block. Considering the allowable unit bearing of oak perpendicular to the grain as 700 pounds, the depth that the block should cut into either the tie or bolster should be $\frac{11{,}010}{700 \times 6 \times 2} = 1.31$ inch, and the block should be made $2\frac{3}{4}$ inches in thickness or depth.

Besides stress investigation the width of the oak block should be analyzed to see whether it has sufficient shearing resistance in a horizontal plane to withstand a lateral stress of 11,010 pounds.

62. In Figs. 31 and 32 are shown two types of heel connection for trusses in which the stresses are very great. It is assumed that the design of the building limits the projection of the lower chord member to that shown in the figure. Since the stresses are high, the bolts must be designed to supply the strength of the joint.

In Fig. 31, the shearing strength on the plane $a\ b$ is 11,500 pounds, leaving $73{,}000 - 11{,}500$, or 61,500 pounds, to be taken by the bolts. In the stress diagram lay off $a'\ b'$, representing 61,500 pounds, and draw a vertical line through b', prolonging it until it intersects a line drawn through a' parallel to the upper chord, at some point c'. The amount of stress in the upper chord to be resisted by the bolts is obtained by measuring the line $a'\ c'$ with the scale employed in laying off $a'\ b'$.

This stress is resisted by a direct pressure on the joint *b c* and by the tensile resistance of the bolts. The stress on the bolts is therefore found by drawing the line *c' d'* perpendicular to the cut *b c*, and the line *a' d'* parallel to the direction of the bolts; then the length of the line *a' d'* will represent the stress to be resisted by the bolts, or 167,000 pounds. If eight bolts are used, it is found by calculation, similar to that already given, that their diameter should be at least $1\frac{1}{2}$ inches.

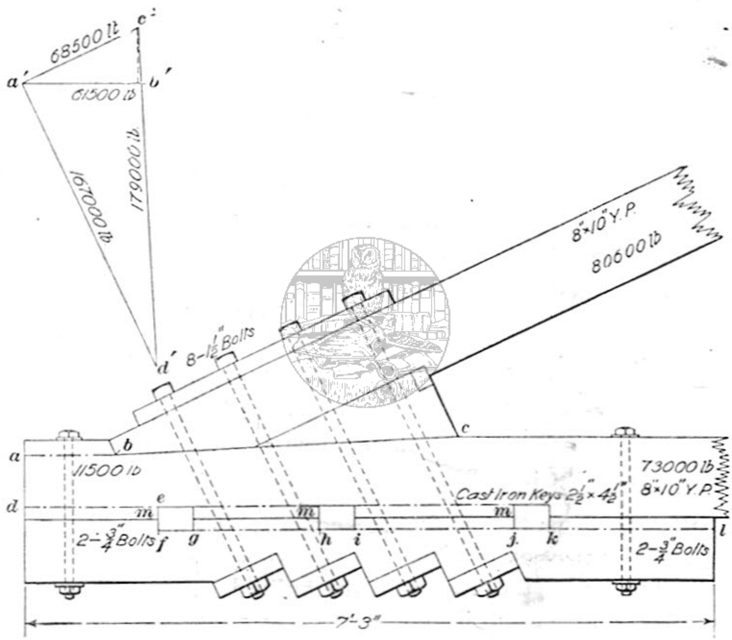


FIG. 31

The bolts transmit their stress through the bolster to the tension member by means of keys, and in designing the detail it is well first to make the calculations necessary in proportioning the keys. In this case the force to be transmitted from the bolster to the tension member is 61,500 pounds. To resist the shear, the tension member must have a length of $\frac{61,500}{8 \times 120} = 64$ inches, or 5 feet 4 inches, and the sum of the width of the keys, if oak is used, should

be at least $\frac{61,500}{8 \times 200} = 38$ inches. As the design does not permit the use of keys of this thickness, resort must be had to keys made of either cast or wrought iron, and whose depth must be such that the pressure of the surfaces at m, m, m does not exceed the allowable bearing value of the timbers on end wood. In Fig. 31 the section represented by the dotted lines withstands the shearing stress, and a portion of the length of the bolster is useless. The length

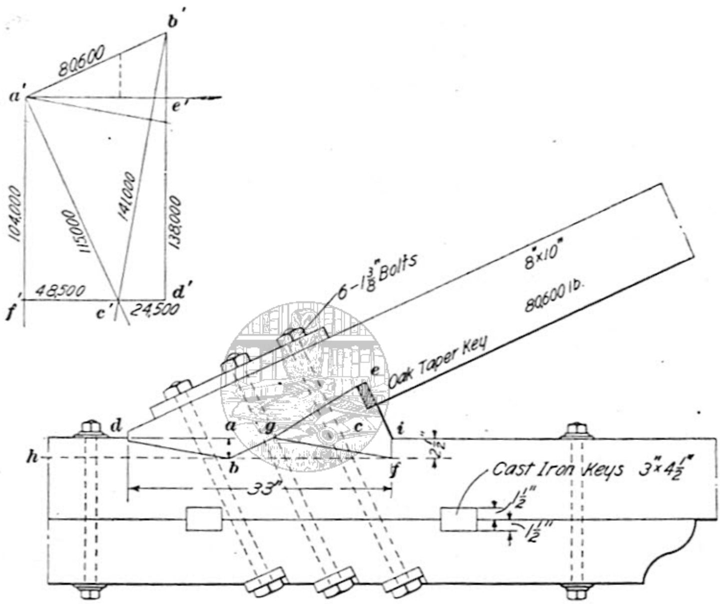


FIG. 32

of the bolster is equal to the sum of the widths of the keys plus the length of the portions of the tension member or tie-member in shear, that is, $g\,h$, $i\,j$, and $k\,l$ plus the projecting end that is not in shear and is made equal to $d\,e$. If these distances are made of equal length, their sum plus the width of the keys, or the length of the bolster, may be expressed by the formula

$$L = a + b\left(\frac{n+1}{n}\right) \qquad (5)$$

in which L = length of bolster;
a = required sum of widths of keys;
b = required length for resisting shear;
n = number of keys employed.

Applying the formula, the required length of the bolster, using three cast-iron keys each $4\frac{1}{2}$ inches in width, is $13\frac{1}{2} + 64\left(\frac{3+1}{3}\right) = 99$ inches, approximately. In the illustration, the length of the bolster shown is 7 feet 3 inches, or 87 inches; if this length is advanced to the amount above found, the factor of safety will be increased. The design shown in the illustration is therefore not as safe as it should be. The width of the keys could be made smaller, but in that case they would tend to turn or roll in the notches. In instances of this kind the constructor must decide whether the length of the bolster shall be limited or made as required.

If the entire stress of 61,500 pounds is equally distributed among three keys, and the allowable unit bearing is 2,000 pounds, the area required for the bearings of the keys against the end grain of the timber, as at m, m, m, is $\frac{61,500}{2,000 \times 3} = 10.25$ inches. The width of the timber is 8 inches, so that the distance which the key should enter the wood is $10.25 \div 8$, or about $1\frac{1}{4}$ inches. The depth of the key will, in consequence, equal $2\frac{1}{2}$ inches.

63. The method just given is quite commonly employed in framing heavy joints, but it is not especially recommended because the whole tendency of the joint is to tear itself apart, and hence the parts are necessarily made very heavy. A superior method of construction is given in Fig. 32, where the joint is worked out in the following manner: A distance $a\,b$ is laid off equal to the greatest cut that may advisedly be made in the tension member, and the point b connected to the point g, where the lower line of the upper chord or oblique member would intersect the upper line of the lower chord. In order to increase the shearing area of the tie-member, or lower chord, a key block whose face is cut

parallel to the face $b\,d$ is inserted at c. The tendency of the joint to slide upwards is resisted by the bolts.

In order to determine the stresses in the several structural elements, the stress diagram is laid off, in which the distance $a'\,b'$ represents the compressive stress in the oblique member. Draw the line $b'\,c'$ perpendicular to the direction of the cut $d\,b$, and $a'\,c'$ parallel to the direction of the bolts. Then the length of $a'\,c'$ represents the stress in the bolts, and $b'\,c'$ the pressure on the cuts $d\,b$ and $g\,f$. The required resistance of the bolts is found to be 115,000 pounds, and hence six $1\frac{3}{8}$-inch bolts will be required to resist this stress. The pressure on the tension member perpendicular to the grain is 138,000 pounds, and if the allowable unit resistance is 600 pounds, the bearing area required is $\frac{138000}{600} = 230$ square inches. The total length of the cut is 33 inches, and its width 8 inches, giving an area of 264 square inches, which is in excess of the area required, so that it is more than safe. The stress of 24,500 pounds parallel to the grain is resisted by a surface equal to the sum of the vertical projections of the cuts $g\,b\,d$ and $i\,f\,g$, or 5 inches. This measurement, multiplied by the width of the timber, will give sufficient area to resist the pressure on the end grain of the tie-member or chord. From the fact that no cuts are made by which direct resistance is applied to the thrust of the upper chord or oblique member, the whole compression in this chord should be considered as above in the calculations for the bolts. The cuts for the keys must then be of such proportions as to resist the whole of the horizontal component of the stress in the bolt. This force, which tends to slide the bolster along the lower chord, is 48,500 pounds, as determined by scale from the line $f'\,c'$ in the stress diagram.

Applying the formula for two keys, the tie-member being of Georgia yellow pine with an assumed allowable unit shearing resistance parallel with the grain of 120 pounds,

$$L = 9 + \frac{48{,}500}{8 \times 120}\left(\frac{2+1}{2}\right), \text{ or } L = 9 + 50.52\left(\frac{2+1}{2}\right) = 84.78$$

inches. The depth of the keys can be found by dividing the entire horizontal stress by the product of the width of the

timber, the number of keys, and the allowable unit bearing resistance of the weaker timber on the end grain. In the calculation following, the distance the key enters either the bolster or the tension member is equal to $\frac{48,500}{8 \times 2,000 \times 2}$ $= 1.51$ inches, or about $1\frac{1}{2}$ inches. Hence, the depth of the key is $2 \times 1\frac{1}{2}$ inches, or 3 inches. The line gb is cut parallel to the upper and lower edges of the compression member in order that no wedge action shall occur and increase the stress in the bolt. Since it is never desirable to have any piece come to what is known as a feather edge, or sharp angle, the point of the compression member is cut blunt, as shown at d. The wedge at e, which is used to bring the block c to a full bearing along the line fg, may be driven tight in case any shrinkage occurs in the timber of the construction.

64. Heel Joints for Trusses With Inclined Legs. In Fig. 33 are shown four detail designs that may be applied to any truss in which the lower chord is inclined. The joints in such a truss should be designed with the greatest care in order to provide rigidity during erection and thus avoid spreading and the consequent thrust on the walls. In (*a*) is given a design in which the angle between the upper and lower chords is very sharp. For this reason it is considered best to extend the upper chord to the bearing block, thus avoiding notching and the shearing stresses that would thereby exist on the tension member. Furthermore, to avoid bending moments at this point, a wrought-iron strap a is located centrally and parallel to the lower chord. The stress in this strap is then equal to the tension in the lower chord and must be proportioned accordingly. The requirements stated tend to reduce considerably the bearing area of the truss on the wall and to throw its center out of line with the line of the reaction, or the center of the bearing block or plate. To increase the area and make the bearing concentric with the downward force, the bolster b is introduced; it is secured in position by the two bolts that hold the joint together.

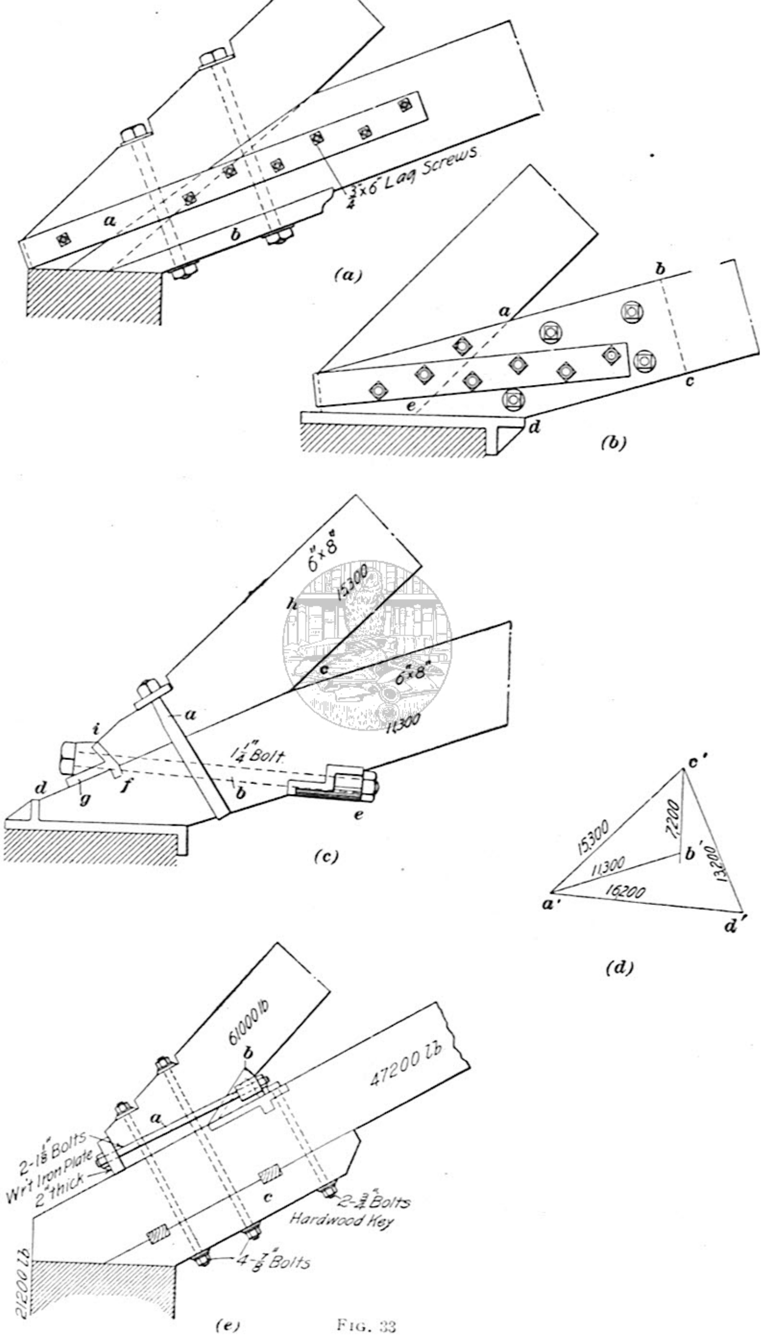

a
b
3/4"x6" Lag Screws
(a)
a
b
c
d
e
(b)
6"x8"
15300
h
c
6"x8"
11300
a
i
1 1/4" Bolt
d
g
f
b
e
(c)
c'
15300
7200
b'
11300
13200
16200
a'
d'
(d)
61000 lb
b
47200 lb
a
2-1 1/8" Bolts
Wr't Iron Plate
2" thick
c
2-3/4" Bolts
Hardwood Key
4-7/8" Bolts
21200 lb
(e)

FIG. 33

In Fig. 33 (*b*), the lower chord is composed of two planks, one on each side of the upper chord member. In order to facilitate the designing of the connection, the strap has purposely been placed out of the center line of the tension member and is carried beyond the upper chord, which necessitates the use of a packing piece, as shown by the shape *a b c d e*. Since the upper chord rests on the wall plate, the stress due to the vertical component of the compression in this member is balanced by the reaction, and no vertical stress is created in the two members adjacent to the bearing.

In the truss shown in Fig. 33 (*c*), the members are of the same width, and the angle between them is not very sharp. The essential feature of the design is that a bolt is used to transmit the stress from the upper to the lower chord. The rafter member rests against a special casting *g*, which is made with a lip *f* to give the joint more rigidity; but it is well to neglect this lip entirely when calculating the strength of the joint. At *e* is shown another special casting that acts as a washer for the bolt and distributes the bearing on the lower chord. The lower part of this casting is stepped to avoid the necessity of making a deep cut in the timber. A strap *a* is used to hold the parts firmly together.

65. The calculations necessary to obtain the strength of the bolt *b* in Fig. 33 (*c*) are as follows: In the diagram Fig. 33 (*d*), lay off, to scale, $a'c'$ parallel to the rafter or oblique member *h*, to represent the stress in the upper chord. Draw $c'b'$ and $a'b'$ to represent the vertical reaction and the tension in the lower chord, respectively, making them parallel to the lines of action of these forces. Since the edge *i f* of the casting is made perpendicular to the direction of the upper chord, this face receives all the compression in the rafter member. The stress in the bolt may be considered as consisting of two forces, one at the end of the rafter and acting parallel with this member, as represented by $a'c'$ in the stress diagram, and the other acting perpendicular to the cut surface *d f c* of the lower chord, and represented by $c'd'$ in (*d*), which is drawn perpendicular

to the face cd in the detail; $a' d'$ is parallel to the axis of the bolt b. The lengths of these lines, measured with the same scale, will therefore give the pressure on the face cd and the stress in the bolt b. The latter is found to be 16,200 pounds, which, divided by 18,000, the assumed allowable resistance of the material composing the bolt, gives .9 square inch as the required area of the bolt at the root of the thread. Since the area at the root of the thread of a $1\frac{1}{4}$-inch bolt is .89 square inch, a bolt of this size will offer the required resistance. The calculations regarding the strength of the timber required are similar to those given under previous examples.

66. In Fig. 33 (e) is given an excellent design for a joint. The tension of the lower chord is taken care of by the bolts a, which are fastened to two castings and pass outside the rafter member. The stress in the bolts is somewhat relieved by the bearing of the oblique member on the casting, as at b, which takes a large amount of the stress in the joint. A bolster c is used to increase the bearing area of the truss on the wall, and its tendency to slide along the lower chord due to the pressure on the lower part is resisted by two keys.

WALL BEARINGS AND CEILING SUPPORTS

67. Wall Bearings.—In wooden trusses, a stone bearing block or wooden plate is usually preferable to one made of cast iron, especially in cases where the truss is exposed to the elements to any degree, as in sheds, since the cast iron rusts and is apt to rot the end of the member, thereby limiting the life of the truss. In setting the stone, it is well to give the upper surface a pitch of about $\frac{1}{8}$ inch to the foot, to prevent water from lying on top of the templet, as it might if there were an exposed eave line or a leaky roof.

68. Ceiling Supports.—To form a **ceiling support** purlins are extended from one truss to the other, usually on the lower chord. These, in turn, support furring strips, to which the ceiling is attached. Ceiling purlins are supported

by special castings and wrought-iron straps in the same manner as roof purlins. Various means are employed in securing the furring strips to the purlins, a few of which are indicated in Fig. 34. In (*a*), a cleat is nailed along each side of the purlin, on which the furring strips rest directly. In (*b*),

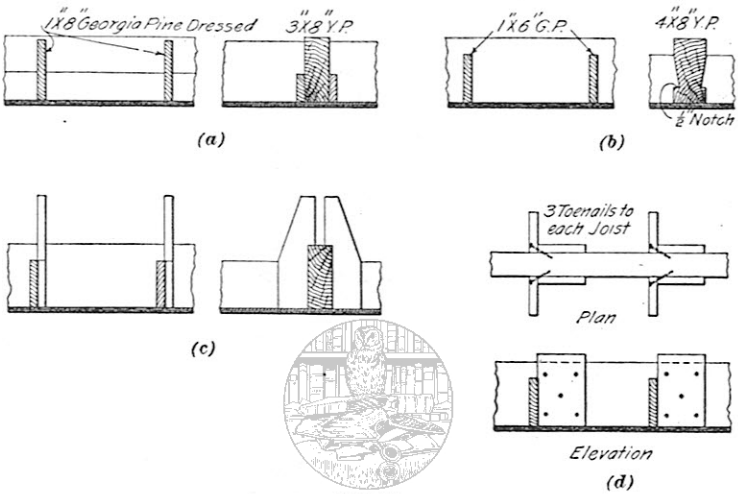


FIG. 34

the furring strips are gained into the purlin itself; this latter method is weaker and more expensive, requiring more work. In (*c*), the strips are hung on the purlins by means of wooden hooks cut to fit over the purlins. In (*d*), the strips are toe-nailed to vertical pieces nailed to the side of the purlin.

COMPOSITE-TRUSS DESIGN

MEMBERS

69. Tension and Compression Members.—In the **composite type** of construction, the upper chord is always made of timber and the tension members of wrought iron or mild steel, while the struts are usually timber, although they may be built up of structural shapes.

The method of calculating the size of the compression members is the same as that given under Timber-Truss Design. The tension members are usually composed of round or square steel bars connected either by pins to special castings or directly to the timbers. Their net area is obtained by dividing the total stress to be resisted by the safe strength of the material per square inch, a factor of safety of 3 or 4 being sufficient in this class of work. Where the net area is taken at the root of the thread, it should exceed the net area required by the calculations by about 15 per cent., to allow for the weakening of the section by cutting the thread, since cutting or scratching the surface of metal slightly reduces its tensile resistance.

70. All tension members should be provided with some means of adjusting their length so that they can be drawn tight, thus causing each rod of a system to withstand its share of the total stress. This adjustment is accomplished by the use of a thread cut on the rods, and in order to avoid weakening the member the ends may be upset, or increased in diameter, so that when the thread is cut on the end, the area at the root of the thread will exceed the size of the rod by 15 per cent. By upsetting the ends of the tension member in this way it is made of uniform strength throughout. This adds to the cost of small bars, but since the strength of any rod is calculated from the smallest area of cross-section, enough material is saved in large rods to compensate for the extra labor required.

71. There are three methods by which the length of tension members may be adjusted—by the use of nuts, clevises, and sleeve nuts or turnbuckles. *Nuts* require a square bearing, so that special castings must be employed when they are used. When the truss is pin-connected, nuts cannot be used; therefore, to tighten the tension members a rod provided with a right-hand thread at one end and a left-hand thread at the other is screwed into the clevis, as shown in Fig. 35. *Clevises* are used when one rod is desired, since their designs are such that they cannot well be used in a

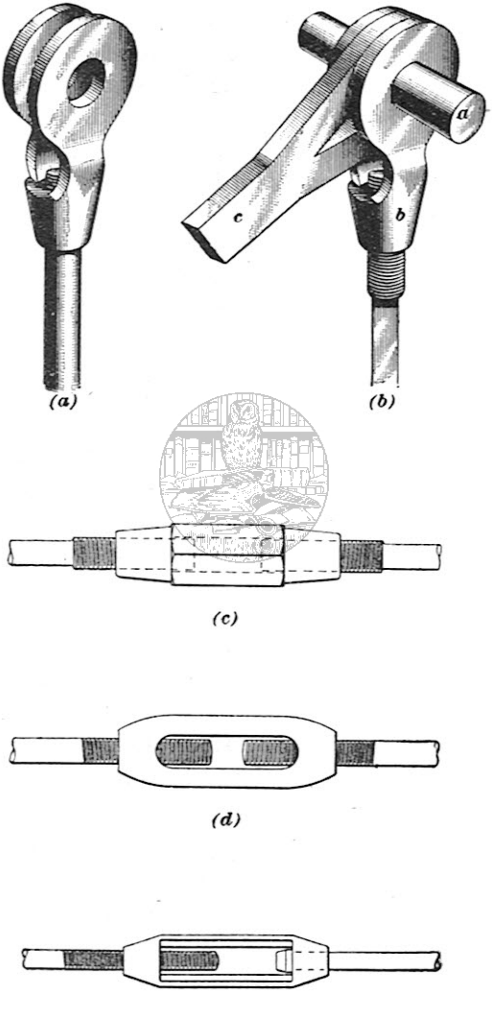


FIG. 35

position requiring two rods. In cases where two tension members are to be fastened to the same pin one of them is provided with a loop that fits between the ends of a clevis on the other member. This construction is indicated in Fig. 35 (*b*), in which *a* is the pin and *c* the loop that is placed between the ends of the clevis *b*. By this means the loads on the pin are balanced.

A *sleeve nut* is illustrated in Fig. 35 (*c*), and *turnbuckles* in (*d*) and (*e*). When either is used the tension member is made in two pieces, usually of equal size, one end of each piece being looped, and the other end upset and threaded. The threads on opposite ends of the turnbuckles should be right- and left-handed, so that when the nut is turned it will screw up on both. A disadvantage in the use of sleeve nuts is that the end of the rod is enclosed and its thread cannot be inspected to ascertain whether the rod is screwed into the nut to the full depth of the thread, while when turnbuckles are employed, the thread of the rod is exposed and its condition can readily be seen. When it is not convenient to cut a left-hand screw, a swivel may take its place, as shown in (*e*). To form the loops given in Fig. 35 (*b*), an iron bar is heated and the end bent back far enough to leave an opening the diameter of the pin, and then welded. This, of course, requires an additional length of bar, varying from $9\frac{1}{4}$ inches for a $1\frac{3}{4}$-inch rod and a $1\frac{7}{16}$-inch pin to 32 inches for a $2\frac{1}{2}$-inch rod and a $5\frac{7}{8}$-inch pin.

CONNECTIONS

72. Joints in General.—Since it is practically impossible to form an absolutely rigid connection between small metal tension rods and wooden compression members in composite trusses, use may be made of *adjustable joints*, or joints in which the members are free to move to a limited extent. In such cases, although the members can adjust themselves to the strain, a great deal of the rigidity of the joint is sacrificed, necessitating the use of strong cross-bracing between trusses.

73. Heel Joints.—In Fig. 36 are given the details of a number of **heel joints** that may be used in composite trusses. In (*a*), the lower chord is raised, as in a church truss, the compression member being inserted between the two lips shown, and held in place by lagscrews. These screws simply hold the casting in place during erection, and are not subject to any strain after the truss is in place. In (*b*) is given a detail of a shoe similar to the one in (*a*),

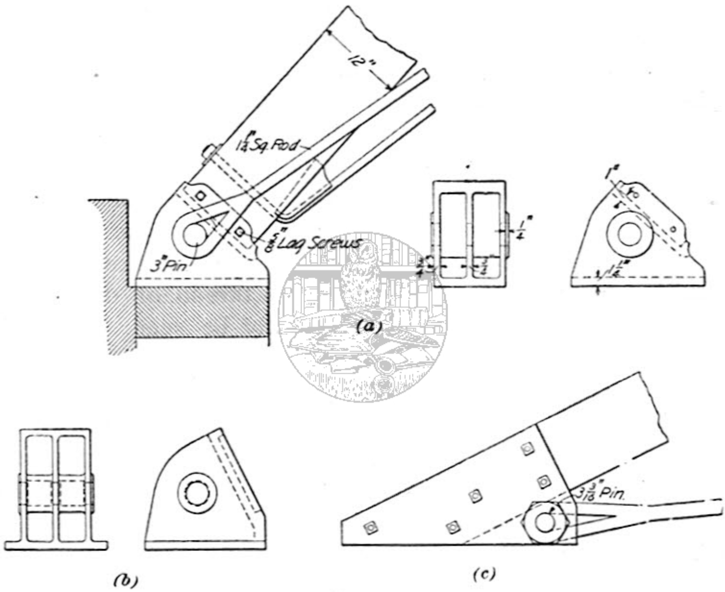


FIG. 36

while in (*c*) steel plates are connected to the wood by bolts, whose strength for both bending and shear should be calculated.

74. Strut Joints.—Many of the **strut joints** given for timber trusses may be employed in composite trusses. For example, methods given in Figs. 11, 12, 13, and 14 may be employed for the upper ends of struts of composite trusses of the Howe form. The joints illustrated in Fig. 37 (*a*), (*b*), (*c*), (*d*), and (*e*) are used to secure struts and ties in Fink trusses. In (*a*), the strut *b* rests against a bolster *d* that

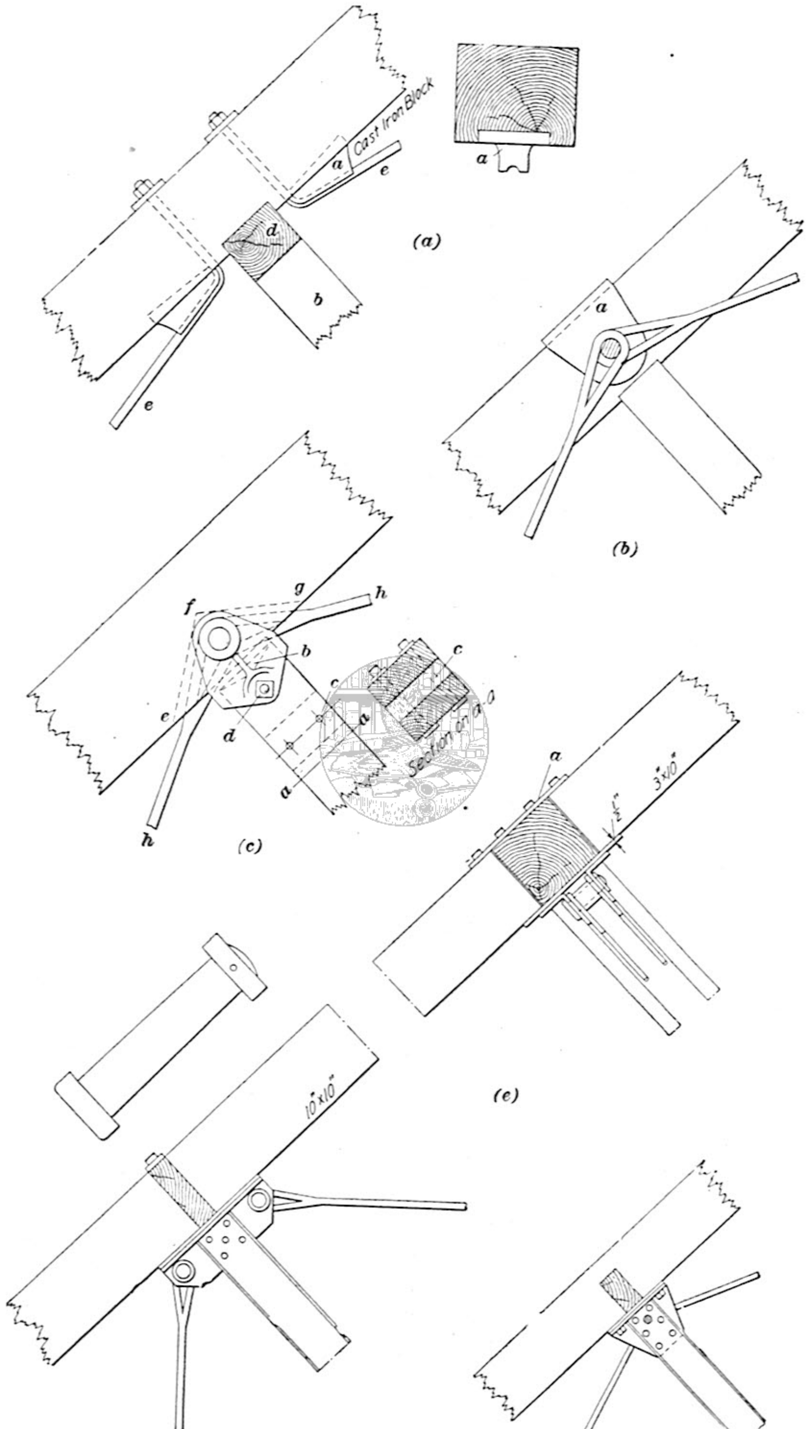

Cast Iron Block
a
e
d
b
e
(a)
a
a
(b)
f
g
h
b
c
c
e
d
a
a
h
Section on a-a
(c)
a
3"x10"
1/2"
(e)
10"x10"
(d)

FIG. 37

serves to support the roof purlins. The rods *e*, *e*, which ordinarily would be connected to a pin, are passed around the casting *a*, and act as camber rods for a portion of the Fink frame. The effect of shrinkage in the wood bolster *d* may be avoided by using a casting in its place.

In Fig. 37 (*b*), the strut is framed directly against the upper chord. To strengthen the joint, a cast-iron stirrup *a* is slipped over the compression member to support the pin at the panel point, and to increase the bearing area. If the casting were omitted, the diameter of the pin would be determined by the compression on the wood of the upper chord. By the use of the casting, the consideration of sufficient bearing for the pin is of secondary importance, the size of the pin being determined by its resistance to shear or bending. Its diameter may therefore be considerably smaller than when no casting is employed.

In Fig. 37 (*c*) is given a pin connection for the same truss, in which the casting used serves not only as a bearing for the strut but also holds the pin to which the tension members are fastened. In this connection the strut and sometimes the rafter members are made up of two pieces of timber separated from 2 to 4 inches, by wooden blocks, the whole being held together by bolts. When the strut is of the section shown, a lip *b* just the width of the space between the ends of the two timbers composing the strut should be provided in the casting. This projection serves to keep the two members apart, though a wooden separator is usually placed adjacent to the casting, as shown at *c*. The bolt *d* holds the casting in place on the end of the strut and thus facilitates erection, besides preventing any possibility of displacement by sudden shock or jar from accidental loads. The rafter member, if solid, must be cut out as shown at *e f g*, to accommodate the ends of the tension bars *h*, *h*, though should the member be composed of two pieces with a space between them, this space is made large enough to accommodate the ends of the bars. On account of the position of these ends on the pin, it is well to provide one with a clevis and the other with a loop, as shown in the detail in Fig. 35.

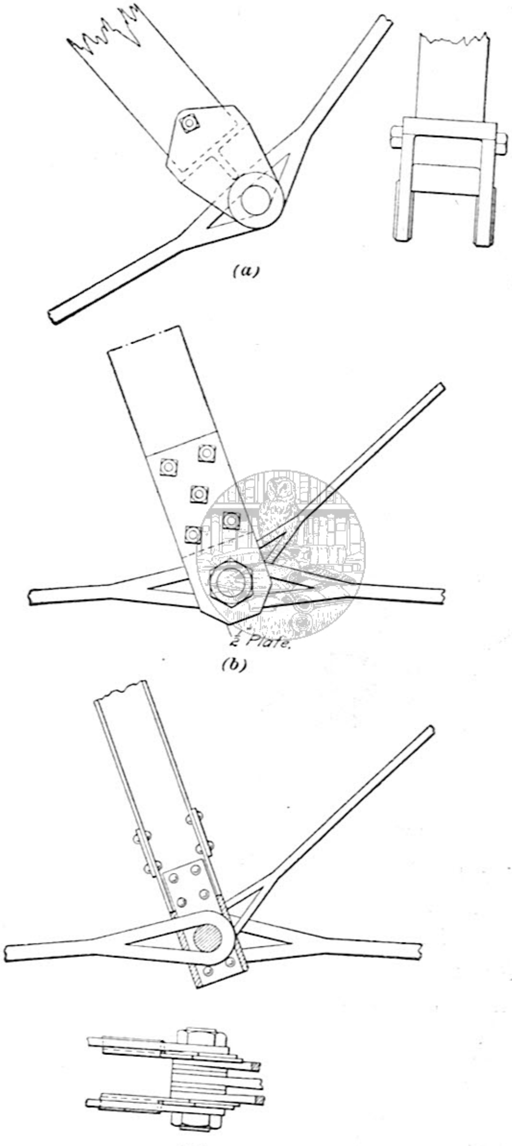


FIG. 38

75. When the strut is of structural steel, such a joint as the one in Fig. 37 (*d*) may be used to advantage, the upper end of the strut serving to hold the pin. In the detail (*e*) two pins are used and the members retain positions corresponding to those in the frame diagram, the necessity of cutting the compression member being avoided without causing the design to loose all the advantages of a pin connection. After the pin is in place a sleeve is slipped on, and secured in place by a taper key driven into a small hole bored through both pin and sleeve. If desired, a split pin may be used, since the purpose of the sleeve is simply to hold the pin firmly to prevent displacement. When two rods are used, they are kept apart by a separator, with which the pins should be provided. The ends of the purlins are strapped to the main rafter member and to each other by means of a piece of iron bar *a*. These purlins are supported by the flat plate that caps the strut.

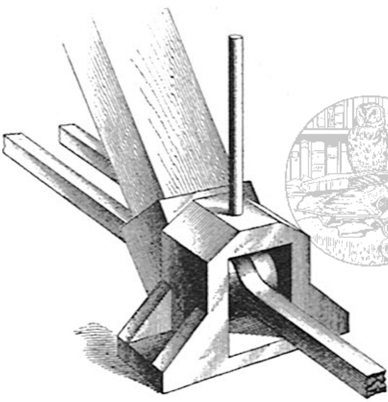
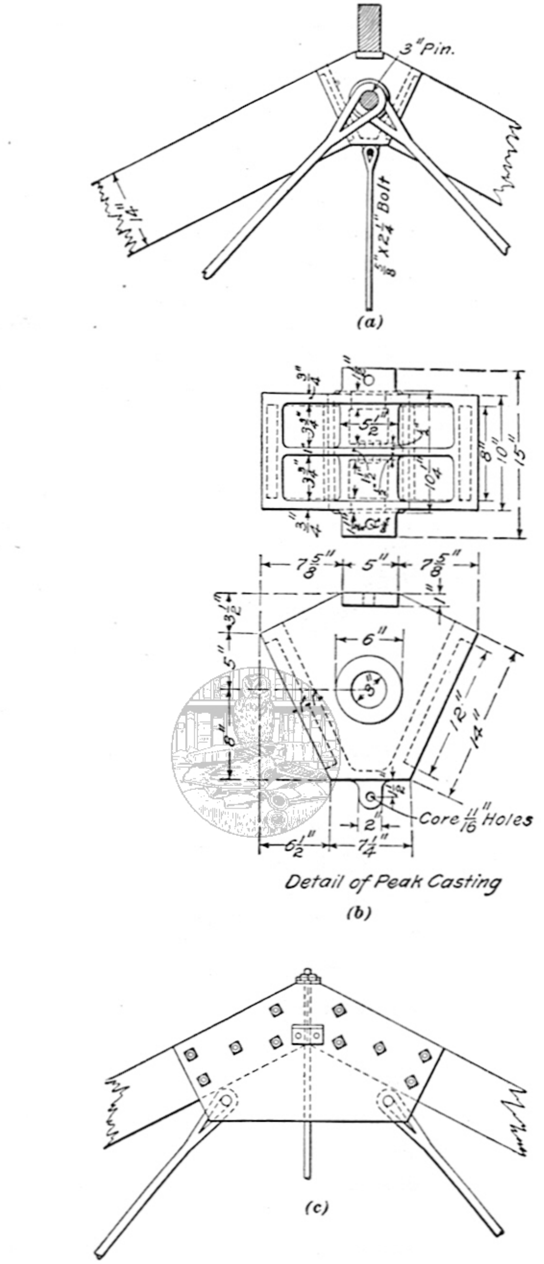


Fig. 39

76. In Fig. 38 are given three designs for the lower end of the strut of a Fink truss. The casting in (*a*) is similar to the one in Fig. 37 (*c*), while in (*b*) plates bolted to the sides of the strut are used, and in (*c*) the strut is built up of two channels having a pin passed through the end. As no new features are here introduced, further explanation is unnecessary.

77. A convenient method of framing the members of a Howe truss is shown in Fig. 39. A cast-iron box, provided with holes for the pin and brackets to receive the ceiling purlins, is used. The strut is inserted in a bearing bracket, and the vertical tension rod passes through the box and is held in place by a nut on the inside of the box. The



(a) (b) (c)

FIG. 40

35—34

tension members are put in position and the pin driven through the casting. The angle of the bearing bracket is altered for the particular panel in which it is to be placed.

78. Peak Joints.—Since compression members cannot be continuous at the peak, some means of holding the pin in place must be adopted in pin-connected trusses. In Fig. 40 is given a detail in which a casting is employed to hold the ends of the compression members and the pin. From (*b*) it can be seen that two lips are formed on the top of the casting to support the ridge pole. In (*c*) the pins are held by two wrought-iron plates bolted to the sides of the compression members. The designs in Fig. 18 are applicable to composite trusses as well as to those composed entirely of wood.

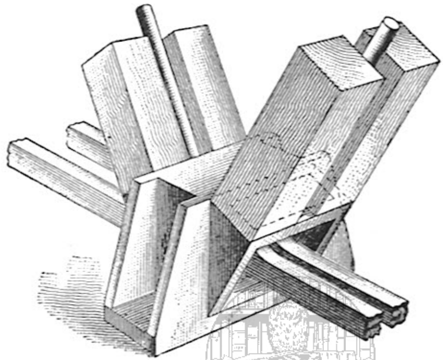


FIG. 41

79. Center Joints.—Some form of casting is usually employed at the **center joints** of Howe trusses. The design shown in Fig. 41 corresponds with the design for the strut joint illustrated in Fig. 39.

STEEL-TRUSS DESIGN

TWO METHODS OF CONSTRUCTION

80. All trusses whose members are composed of steel or wrought iron belong under this subdivision. Formerly wrought iron was extensively manufactured, since nearly all the structural shapes were made of it, but on account of the superior strength of steel and the cheapness of its manufacture since the introduction of the Bessemer process, as

well as the convenient shapes obtainable, it readily adapts itself to truss construction, and hence has now almost superseded wrought iron in the market.

81. Steel trusses may be constructed with either riveted or pinned joints, the former being preferable for short spans, because the several members may be riveted together in the shop and the truss shipped in sections. Such frames are put together with ease and rapidity, and since the field work is greatly lessened and the shop work is cheaper than that done on the field, the total cost is decreased. They have the additional advantage of being stiffer laterally than pin-connected trusses, since the joints do not give. In trusses of longer spans, the members are necessarily of such great length that if two were joined together shipment would be almost impossible, and the work in the field would be greatly increased. In such cases it is well to use pin-connected joints, fitting the parts so carefully in the shop that they can be readily assembled in the field.

It is customary to add $\frac{1}{32}$ inch of metal to all surfaces in trusses exposed to the action of corroding gases. Pin-connected trusses are preferable in many instances since their tension members are usually round or rectangular in section, thus exposing less surface to corrosion than do angles figured to withstand the same tension.

MEMBERS

82. Riveted Tension Members.—The shapes most commonly employed for **riveted tension members** in steel-truss connections are angles, flat bars, and, occasionally, channels. Angles, being stiffer than flat bars and having less section than channels, are used to the greatest extent, usually being placed in pairs, back to back, as shown in Fig. 42 (*a*), which is a section through *a a* in (*b*).

To provide greater rigidity and prevent the angles from striking against each other during vibration, small washers, called *separators*, are placed at intervals of $2\frac{1}{2}$ or 3 feet

between the angles, as at b in (b). These consist of small pieces of plate, round or square, having holes of the same diameter as the rivet holes punched through the center, and should be of the same thickness as the gusset plates to which the tension members are fastened, and held in place by rivets driven through both angles and separators.

83. The building laws of some cities provide that when an angle is connected by but one leg, the strength of that

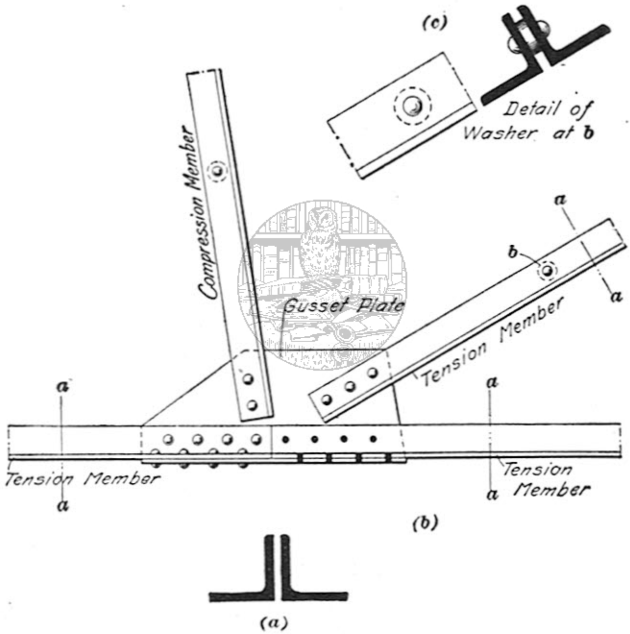


FIG. 42

leg only is to be considered when figuring the resistance of the angle, it being claimed by some that as the angle is eccentrically loaded, it is liable to tear through the rivet holes, as would a piece of perforated paper. At any rate, it is always preferable to connect both legs of the angles by splice plates, cover-plates, or splice angles, which connections will be fully explained later.

84. In riveted trusses, flat bars are seldom adopted for the lower chord, but they may be used for tie-members in cases where there is no liability of the stress being changed to compression by the eccentric loading of the truss from the action of the wind or other causes. In calculating the strength of flat bars as tension members, the whole section may be considered and the net section obtained by deducting for rivet holes. The only advantage gained by using these flat bars is that less riveting is required than when the members are composed of angles placed back to back, as there is no projecting leg to connect by means of plates or splice angles. When the lower chord supports a floor load distributed over its length, it is well to use channels placed back to back, and in calculating the size of the member, to consider its ability to resist bending as well as tensile stresses.

85. Pin-Connected Tension Members.—When there is no danger that the stress in the tension member of a pin-connected truss will change to compression, round, square, or flat bars of either iron or steel may be used; but if the stress is liable to change, channels used in pairs are preferable. The round and square bars are employed as tension members only when the stresses are comparatively light, their use and the means of adjustment having been fully treated under Composite-Truss Design.

When the span of a truss is long and the stresses to be resisted are great, pin connections are commonly adopted, and a suitable tension member is formed by a flat bar of rectangular section, such as is illustrated in Fig. 43. For the

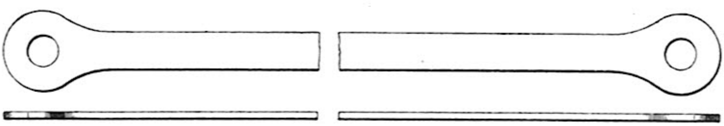


FIG. 43

sake of economy it is well to make the panels of equal length, and by varying the number of bars in the panel, it is often possible to use bars of the same size for several different members. All machine work on these tension bars,

such as boring the pinholes, must be done with the greatest accuracy, and some engineers specify that the distance between centers of the pinholes in the several bars shall be exact when the temperature of the metal is 64°. A more common requirement, however, is that the pinholes fall directly in line, one above the other, when the bars are placed in a neat pile, and that a pin having a diameter $\frac{1}{32}$ inch less than that of the hole will pass through them all without forcing. This insures bars of proper length and pinholes of uniform size, the importance of which can readily be seen when it is considered that there is no way of adjusting the length of the bars.

To facilitate the work and obtain accuracy in the shop, it is customary to clamp a number of these bars together and drill the pinholes for all at the same time. The enlargement at the end of the bar should be obtained by upsetting, and no weld should be permitted throughout the member. The diameter of the upset end should be such that when the hole is cut for the pin, the area on a transverse line through the pinhole will be at least 33 per cent. more than that of the rod. The pinhole should have $\frac{1}{32}$ inch clearance and must be exactly at right angles to the plane of the rod. When considering the order in which several bars are to be placed on a pin, the arrangement adopted should be the one that causes the least bending moment on the pin, and a space of no less than $\frac{1}{2}$ inch should exist between tension bars in order to provide for painting.

Whenever practicable, the tension members should lie in a plane parallel to the plane of the truss, but when this is impossible the divergence from the plane of the truss should not be more than $\frac{1}{8}$ inch to the foot.

86. Channels used as tension members are usually latticed together and made long enough to extend beyond the pin. It is preferable to avoid cutting away the flange at the ends of the channels, but when such a practice becomes necessary the web should be reenforced with pin plates riveted to it, so that the strength of the member will not be

diminished. These pin plates also serve to reduce the unit pressure on the pin to the allowable limit, or even lower; they should be long enough to properly distribute the stress to the rivets connecting the plates to the channel, and should extend no less than 6 inches within the tie-member so as to provide for at least two transverse rows of rivets. The net section at right angles to the axis of the member and through any pinhole in these riveted tension members,

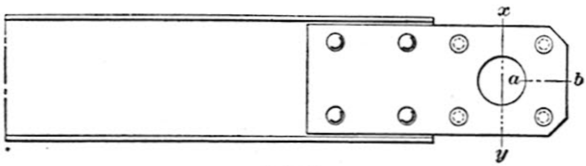


FIG. 44

as along the line xy, Fig. 44, should be at least 40 per cent. greater in area than the net section of the body of the member, and the net section along the axis ab should be at least 70 per cent. of the net section along xy. Members of this character should not be spliced unless absolutely necessary, in which case the splice must be proportioned to the full strength of the member instead of to the stress to be resisted.

87. Riveted Compression Members.—The sections usually employed for compression members in riveted trusses are given in Fig. 45 (*a*), (*b*), (*c*), and (*d*). In (*a*), two angles are placed back to back and held apart by separators *a*. These separators are of greater importance in compression than in tension members, as they cause the angles to act in unison, increasing the radius of gyration of the section and consequently the strength per square inch. This can be clearly seen by referring to any table giving the properties of angles. Were the angles shown in (*a*) to act separately, their least radius of gyration would be about the axis xx and would be equal to .86, while when separators are used the least radius of gyration of the section is about the axis yy, and is 1.71, an increase of nearly 100 per cent. In compression members, separators are usually placed along

the whole length of the member at any distance not exceeding eight times the shortest leg of the angle used. The section in (*a*) is most commonly used, as it can readily be put together and because it affords a section of small area for roof trusses whose stresses are light.

When the loads are heavy, or a load must be placed between the panel points, the section in (*a*) may be strengthened by the introduction of a web-plate between the angles, as in (*b*), or channels placed back to back, as in (*c*), may be substituted. When the form in (*b*) is used,

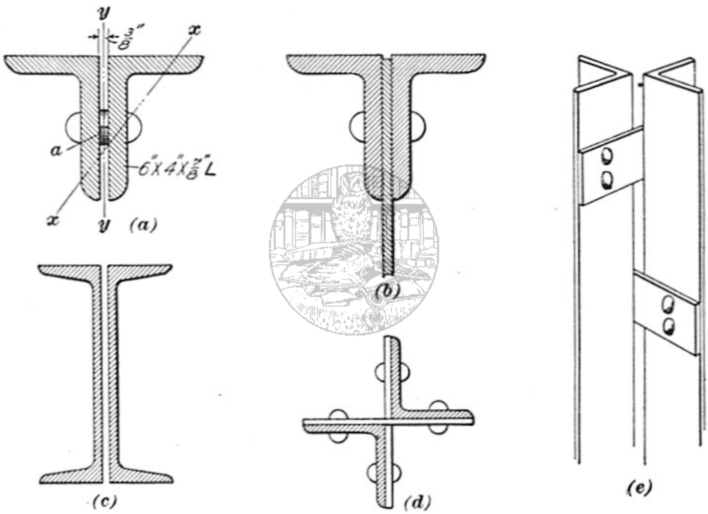


FIG. 45

rivets should be placed along its length at intervals of not more than sixteen times the thickness of the angle, and in no instance should the pitch be more than 6 inches.

88. The section usually employed for struts is the same as that for compression members shown in Fig. 45 (*a*), although that indicated in (*d*) is used to some extent. In this case the angles are placed diagonally opposite each other and are fastened together by means of plates, two rivets to an angle in each plate, as shown in the elevation in (*e*). When the upper chord must be heavy or is subjected to cross-bending

stress, channels placed back to back are suitable, and they may also be used to advantage for long struts in riveted work. When the load carried by the strut is light, one angle is sometimes used, although ordinarily this is poor practice, as the stress is imposed on one side of the truss. When a single angle is used as a compression member, the legs of the angle should be equal, in order to have the greatest possible radius of gyration with the use of the least material.

In splicing a compression member composed of angles placed back to back, it is advisable to connect both legs, as in tension members. In no case should the radius of gyration of the section selected be less than $\frac{1}{150}$ of the length.

When calculating the strength of compression members the full area should be considered, no deduction being made for rivet holes, since shop rivets are commonly employed along the length between panel points, and these, when properly heated, completely fill the holes and bear their share of the compressive stress.

89. Pin-Connected Compression Members.—These members are usually made up of channels placed back to back for the chords, and with the backs on the outside for the struts. By this arrangement the struts can be conveniently entered between the backs of the chord members, thus avoiding the necessity of cutting away the flanges at these joints.

Sometimes the upper chord has a cover-plate on top and is latticed at the bottom. This, however, is unnecessary in most roof trusses unless the span is so great that it is desirable to increase the area of the section in this way. The distance between channels should be such that the radii of gyration about the two axes are equal, although the great advantage gained by keeping the truss of uniform width throughout should not be sacrificed for this purpose. Whenever possible, the width of the truss should be such that the radii of gyration of the largest compression member are equal about both axes.

Although channels are used almost entirely in trusses of long span, in the shorter spans, where the members are not

required to withstand great stress, the struts are composed of two angles placed back to back.

90. As in tension members, pin plates are used whenever a compression member connects with a pin, as shown at *a*, *a* in Fig. 46. The pin plates serve the double purpose of reenforcing the section of the pin and reducing the unit bearing stress on the metal adjacent to the pinhole. Where the load is light, one pin plate is used, but for heavier loads it is preferable to place one on each side of the web of the channel. When it is necessary to place a tension member next to the compression member, countersunk rivets may be used, and in such cases, the plate in which the rivets are countersunk should be at least $\frac{7}{16}$ inch thick. The channels and pin plates should extend far enough beyond the pin to permit at least two rivets to be driven to hold the pin plates and web together. In figuring the strength of these compression members, the usual column formulas should be employed. When it is found necessary to splice a compression member, the splice is made equal to the full strength of the member, and no reliance is placed on the ends of the channels abutting.

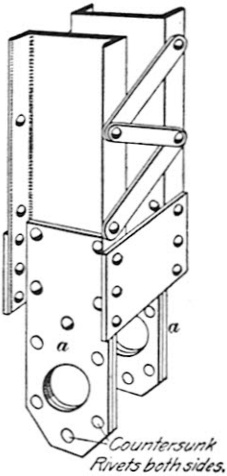


FIG. 46

91. Splices in Riveted Trusses.—The simplest form of splice used in joining tension and compression members consists of a plate riveted between the legs of the angles. No figure is given to illustrate this as it is the same as that given in Fig. 47 (*a*), except that the cover-plate is not used. The strength of this splice is limited, however, as it connects but one leg of each angle, and hence, where greater stress is to be resisted the form shown in Fig. 47 (*a*) may be employed with advantage, since by its use the load may be transferred concentrically to the next member.

By using splice angles, as in (*b*), there can be secured a

strong and efficient splice, which may be still further strengthened by riveting on a cover-plate. If the angles are not of the same thickness the thinner must be built out to the

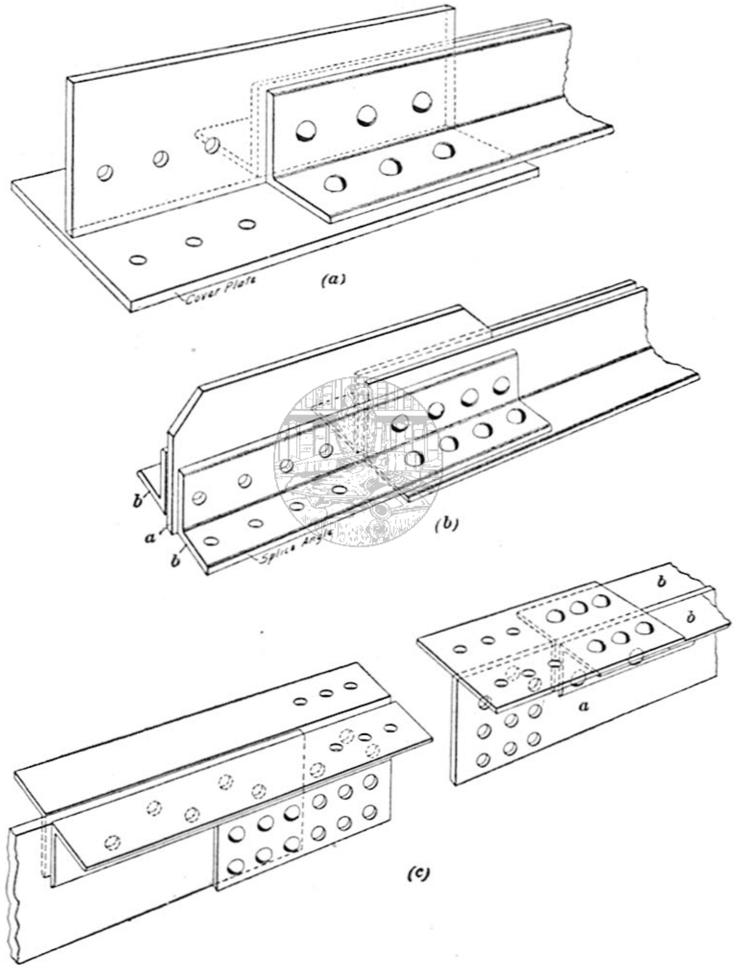


FIG. 47

size of the larger by using packing plates, although if they do not vary more than $\frac{1}{8}$ inch the splice angle may be swaged or bent to take up this difference.

When the section illustrated in Fig. 45 (*b*) is adopted, it is well to use the splice shown in Fig. 47 (*c*). This is made by carrying the web *a* beyond the ends of the angles *b*, *b*, thus forming a rigid connection. In order to avoid reduction of area at the splice, it is necessary to introduce splice plates on either side of the web, and sometimes a cover-plate also, as indicated in the figure.

The plates and angles used in making a splice must take care of the entire stress, the strength of the members themselves being entirely disregarded, and the strength of the splice must be proportioned to the full strength of the tension or compression member, and not simply to the stress that they may be called on to resist. This makes the strength of the member uniform throughout its length.

92. The strength of a splice may be figured as follows: When a tension member is being considered, the strength of the section should be computed, and if a splice plate is to be used the number of rivets required is determined by dividing the strength of the tension member by the bearing value of one rivet, or by the shearing value of one rivet in double shear if the tension member is made up of two pieces; these two values should be ascertained and the smaller one used in determining the number of rivets. The length of the splice plates or angles is readily obtained when the number of rivets and spacing are known. The splice plate should be of such thickness that its strength in web bearing is not exceeded by the strength of the rivet either in double shear or in bearing value on the rolled shapes that compose the tension member. The net area of the required splice must be made equal to the net area of the tension member. The rivets should be so distributed that the approximate center of gravity of all, taken collectively, will be near the center of gravity of the rolled shapes in the tension member.

93. The method of designing the splice for compression members in a roof truss is similar to the above with the exception that the strength of the member is figured for the full area of the angles, the splice plates being made equal in

area. The following example clearly illustrates the principles involved:

EXAMPLE.—What is the tensile strength of the connection shown in Fig. 48?

SOLUTION.—Using $\frac{3}{4}$-in. rivets, the net allowable area of the angle would be $3\frac{1}{2} \times \frac{3}{8} - \frac{7}{8} \times \frac{3}{8} = .984$. Assuming 14,000 lb. per sq. in. as the tensile strength of the material composing the member, the strength of each angle is $.984 \times 14,000 = 13,776$ lb., and of two angles is $2 \times 13,776 = 27,552$ lb. If the unit shearing value of the rivets is 10,000 lb., the strength of one rivet in double shear is equal to twice

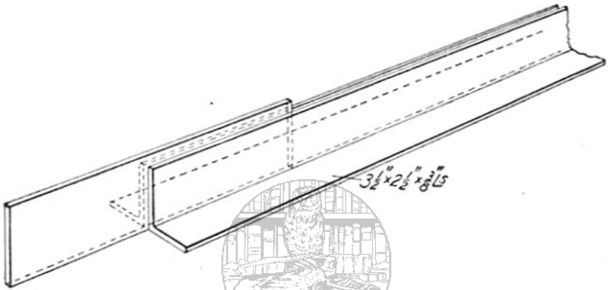


FIG. 48

the product obtained by multiplying its area in cross-section by 10,000. The area of a $\frac{3}{4}$-in. rivet is .4418 and its strength in double shear is, therefore, $.4418 \times 10,000 \times 2 = 8,836$ lb. The number of rivets required is $\frac{27552}{8836} = 3$ rivets. The required thickness of the plate, allowing 20,000 lb. per sq. in. for the bearing value, is $\frac{8,836}{20,000 \times \frac{3}{4}} = \frac{9}{16}$-in. plate. Since the bearing of the rivets in the angles is $\frac{3}{8}$ in., it is evident that they are sufficiently strong. The strength of the connection is therefore 27,552 lb. Ans.

94. Riveted Strut Joints.—The number of structural shapes available for steel trusses limits the variety of forms for strut joints, so that those shown in the figures accompanying the text fully illustrate the principles involved in their design. The general remarks that were given concerning wood and composite trusses apply also to the design of structural strut connections.

The designer should endeavor to so place the angles or flats composing the truss that their center of gravity corresponds with that of the frame diagram. The number of

rivets required to resist the stress in the member is then calculated, the strength, both in shearing and bearing, being analyzed to determine which is lower. In cases where the chord member continues through two panels, the number of rivets required is determined by the difference in stress between the two panel lengths.

95. The ideal position for rivets is along the neutral axis of the angle. However, this is impossible in actual work, because the center of gravity of an angle section is so near the back that there is not room enough to form the rivet head. It is not customary, therefore, to place the rivet hole less than $1\frac{3}{8}$ inches from the back of the angle when a $\frac{3}{4}$-inch rivet is used.

After determining the number of rivets required in each member meeting at a joint, the size and shape of the plate depends on the spacing adopted. The distance between centers should never be less than three times the diameter of the rivets, but never more than 6 inches. The shape of the plate may be altered by varying the spacing of the rivets, which must frequently be done for economy, or for the sake of a better appearance. It is often found that if the shape of the plate is made to indicate the shape of the panel of the truss in which it is placed, the general effect will be more pleasing.

The simplest form of a strut joint in a structural steel truss is shown in Fig. 49. The dot-and-dash lines indicate the neutral axes of the angles, and as can readily be seen, they intersect at the panel point. The edges *g h*, *f j*, and *f g* have been cut parallel to *b a*, *c a*, and *d e*, respectively, which gives the plate a finished appearance and adds character to the design.

96. The following method for determining the number of rivets in the strut connection, Fig. 49, is the one commonly employed for such work. The size of the rivets and the amount of stress they withstand must first be determined. Since roof trusses are not so liable to be subjected to sudden shocks or loads suddenly applied as are trusses

used in railroad bridges, a large allowable unit stress may be assumed.

It will be noticed that there are some field rivets, or rivets driven while the work is being erected, used in this connection. In conservative practice field rivets that are included in the design are assumed to have five-sixths of the allowable shearing value of shop-driven rivets. The reasons for this are that the work cannot be done as carefully on the field as in the shop, the holes frequently are not in alinement, and the heated rivets must often be carried so far that

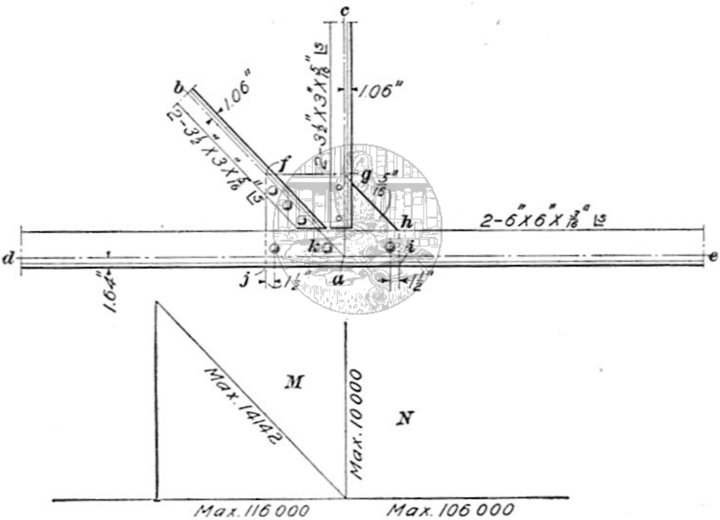


FIG. 49

they are cooled too much to be driven successfully. Other reasons for the inferiority of field rivets are that the machine used in driving them is very light, and that in some cases they must even be driven by hand. When considering the bearing value of the rivets on the plates or rolled shapes, no distinction need be made between shop and field rivets, from the fact that the defects due to field riveting are in the rivets themselves, and the field rivets have as good bearing as those driven in the shop.

The allowable strength of $\frac{3}{4}$-inch rivets, based on material having an allowable unit tensile value of 15,000 pounds, is given in Table II.

TABLE II

Rivet				Plate		
Diameter Inch	Kind	Shear		Thickness Inch	Bearing	
		Single Pounds	Double Pounds		Ordinary Pounds	Web Pounds
$\frac{3}{4}$	Shop	4,788	9,577	$\frac{5}{16}$	4,570	6,094
	Field	3,990	7,980			

The maximum stress in the member $c\,a$ is 10,000 pounds, but though the rivets used are field-driven and in double shear, the bearing value of 6,094 pounds must be taken, as it is less than the double shearing value of a field rivet. From this it is evident that two rivets are sufficient for the connection, while the member $b\,a$, having a stress of 14,000 pounds, will require three.

The number of rivets used for the lower chord should be sufficient to provide for the difference in stress between the portion of the chord at the right of the connection and the portion at the left, which stress is equal to the algebraic sum of the horizontal components meeting at the connection. The difference in stress between the two portions of the tie-member is, in this instance, 116,000 − 106,000 = 10,000 pounds, so that practically two rivets are required. But since the maximum pitch of rivets is 6 inches and the two rivets are placed so near the ends of the plate that there is a space greater than 6 inches between them, another rivet should be used at the center, as shown at k.

97. In cases where the lower chord is not continuous, as in Fig. 50, it is preferable to connect both legs of the angle, not only to give greater strength to the splice, but

also to provide greater lateral stiffness for the truss. The stresses in the several members being given in the figure, the number of rivets required may be calculated as in the following example:

EXAMPLE.—What number of rivets will be required in the connection shown in Fig. 50, if $\frac{3}{4}$-inch rivets are used throughout the joint?

SOLUTION.—The values for rivets and plates given in Table II are used in this case also, but the number of rivets in the connection of the member *d o* must be determined in a different manner from that just shown. Here it is convenient to place three rivets through the vertical legs of the angles; therefore, the strength of these three rivets must be determined and also the remaining stress to be provided for by the rivets in the flanges or horizontal legs of the angles. The strength of the three rivets in the vertical leg of the angle is equal to $3 \times 6{,}094$ lb. = 18,282 lb., so that if the total stress to be resisted is

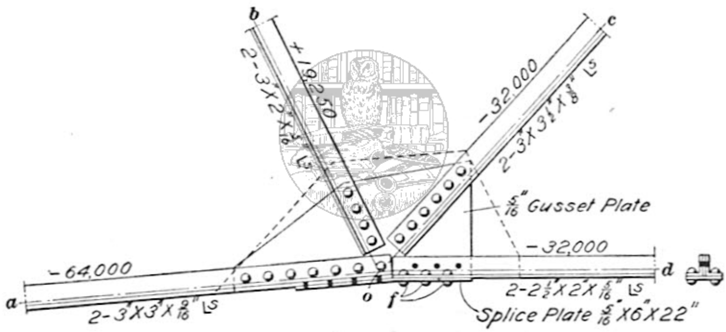


FIG. 50

32,000 lb., the balance to be sustained by the rivets in the flanges is equal to 32,000 − 18,282 = 13,718 lb. These rivets are in single shear and ordinary bearing. The single shearing values for $\frac{3}{4}$-inch shop- and field-driven rivets are 4,788 and 3,990, respectively, while the bearing value is 4,570. Since the lowest of these values must be considered in the analysis, and as the rivets *f* under discussion are shop-driven rivets, their strength will be taken at 4,570 lb. Then, as the total stress that these rivets must resist is 13,718 lb., three rivets are required, but to make the connection symmetrical four rivets must be used, two on each side. Since the member *d o* is connected to *a o* by the $\frac{5}{16}$-inch splice plate, it would be well, in order to realize the full strength of the splice plate, to introduce two additional $\frac{3}{4}$-inch rivets in the lower flange of the angle, so that six rivets, three on a side, are used in this connection. Ans.

Since the rivets are in double shear, and the gusset plate is only

$\frac{5}{16}$ inch thick, the shearing strength of the rivets will not be realized, and their bearing value on the plate must be used in the calculations. This is evident, for the allowable web-bearing value of a $\frac{3}{4}$-inch rivet in a $\frac{5}{16}$-inch plate is 6,094 lb. This value will govern the number of rivets, because the smallest angle in the connection capable of bearing 4,570 lb. is $\frac{5}{16}$ inch in thickness, and as there are two angles, one on each side, the bearing value for one rivet in ordinary bearing will be 9,140 lb. When the double shearing value of the rivets alone is considered, the number of rivets required for the member *b o* is 19,250 ÷ 6,094, or 4, and the number required for the member *c o* is equal to 34,000 ÷ 6,094, or 6.

Similar calculations may be made for the number of rivets in the member *o a*, with the result that this connection is designed as shown.

The joint whose analysis is given above is used in a Fink roof truss, the shape of the plate being determined by the

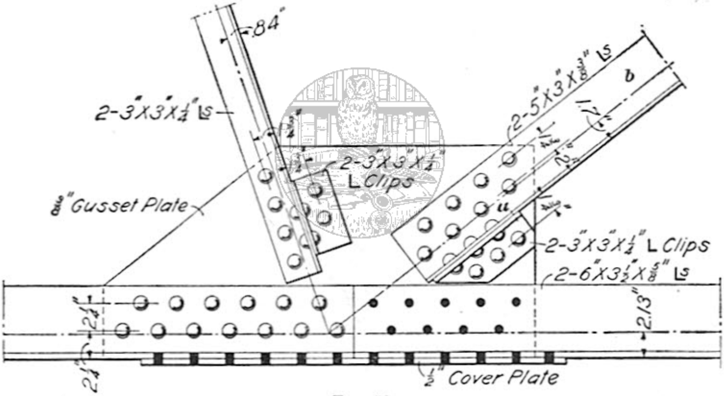


FIG. 51

number and spacing of the rivets, the rivets in this case being placed as close together as possible. By lengthening the gusset along the center line of the member *o b* and making the cuts parallel to the members of the truss, a design can be obtained that might be considered superior to that shown in Fig. 50. This change of design is indicated by the dotted lines, although the effect could more readily be noted if the whole frame diagram were given.

98. When the stresses are high the angles may be connected to the gusset plate by means of angle clips, as in Fig. 51. It is of advantage to make such a clip and the

angle it connects of the same size, and to use the same number of rivets in each, or better still, to so arrange the rivets in both angle and clip that their moments about the center of the connections are equal. By this method the rivets share the strain equally, for the neutral axis of the angle is concentric with the line of resistance of the rivets. This point is demonstrated in the following example:

EXAMPLE.—In Fig. 51, is the load on the member *b* concentric?

SOLUTION.—The distance of the center of gravity from the back of the $5'' \times 3'' \times \frac{3}{8}''$ angle is 1.70 in., and the distance from the three rivets shown in the angle clip to the back is 1.75 in. The moment of these rivets about the axis of the member may then be taken in units of rivets multiplied by the distance from their center line to the center of gravity of the angles forming the member. The moment of these three rivets in the clips will equal $(1.7 + 1.75) \times 3 = 10.45$. The distance from the first row of rivets in the $5'' \times 3''$ angles to the back of the angles is 2 in., and the distance from the second row to the same place is 3.75. The combined moments of these two rows of rivets is $4 \times (2 - 1.7) + 4 \times (3.75 - 1.7) = 9.40$. This shows that the moments of the several rows of rivets about the axis of the member nearly balance each other and thus insure a concentric load on the member. Ans.

It will also be noticed that if it were necessary to place another rivet in the angle clip, as at *a*, an extra rivet would have been driven in order to realize the full strength of this rivet, because for each rivet driven in the outstanding leg of the angle clip, one rivet should be driven in the flat leg.

99. Purlin Connections.—Fig. 52 shows the usual method of connecting the struts and ties when the upper chord is made of two angles and a web-plate, and also gives an excellent method of supporting the roof purlins. The purlins on such a roof are from 4 to 6 inches in width, so that by placing the angle brace *a* 2 or 3 inches below the panel point, the load is brought centrally over the joint, as it should be. This brace is composed of two vertical angles riveted between two horizontal angles, or bent plates cut to shape, the latter in turn being riveted to the upper chord. Provision is made for bolting the purlin to the clip by leaving open holes in the vertical angles.

100. **Purlins** are usually made of angles, **Z** bars, or **I** beams. *Angle purlins* are used with advantage when loads are light, and the connections are made so that the center line of the load falls in line with the panel point, as in Fig. 53 (*a*), an angle clip being used to hold the angle securely to the frame. To give greater stiffness to the design angle purlins should always be placed with the flange pointing up the slope.

The **Z** bar is best adapted for use as a purlin, since its shape gives great strength. It should not be placed as shown by the dotted lines at *m* in Fig. 53 (*b*), since less strength would be developed and a groove would be formed between the web and the lower flange, in which water of

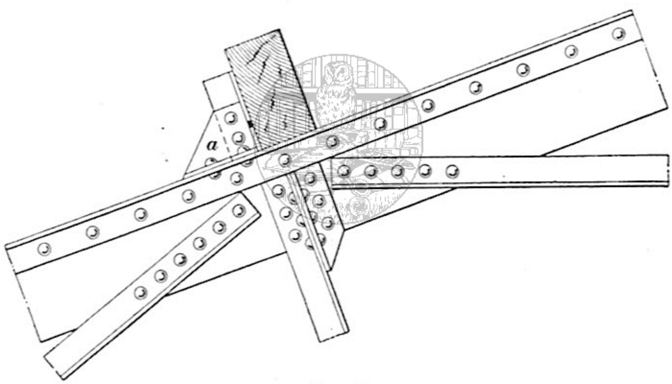


FIG. 52

condensation could collect if there were any leakage whatever from the roof. The connection to the frame is made by riveting through the lower flange, and if two **Z** bars abut at a truss, further connection is afforded by riveting a splice plate to both webs of the **Z** bars, and greater strength may be gained by riveting the upper chord, as shown dotted in the figure, although this is not absolutely necessary.

I beams may also be used as purlins, but are not as efficient as **Z** bars. Connections may be made by riveting through the lower flange to the upper chord of the truss, and they are further secured by a splice plate connecting

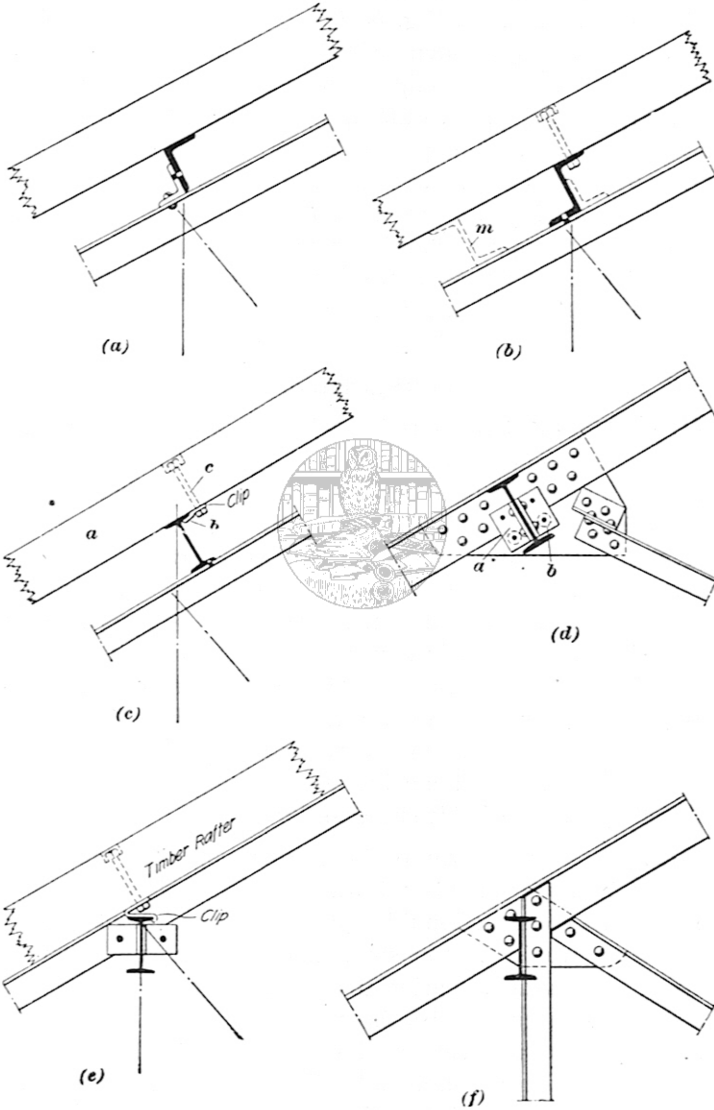


Fig. 53

two abutting purlins. A detail of an I beam employed as a purlin is illustrated in Fig. 53 (*c*); here it can be noticed that the secondary wooden rafters *a* are held to the purlins by means of a bar-iron clip *b* and a bolt or screw *c*. The I beam may be riveted to the upper chord so that the upper flange is on a plane with the upper chord by using two angle clips, as in (*d*). In this case, the clips are usually riveted to the I beams in the shop, and open rivet holes are left in the upper chord for field connections. In (*e*) and (*f*) are given two methods for connecting I beams when the purlin is placed in a vertical position.

CONNECTIONS

101. Heel Joints.—It is as important in steel trusses as in those composed of wood that the center lines of the various members intersect, and that the wall bearing be directly under the panel point and symmetrically placed with respect to that point. **The heel joint** is even of greater importance in steel construction than in wood, especially in trusses of large span, since the stresses to be provided for are much greater than in any type described. The joint at the heel must be figured not only for the tension and compression in the chords meeting at this point, but also for the shear and bending due to the reactions. In large trusses the shear is great, and considerable ingenuity is sometimes required to provide for it successfully.

102. A simple form of heel joint could be made by carrying the rafter member down to the wall plate, but since in steel-truss construction this upper chord is made of angles, the bearing would be too small and some means must be adopted to provide a bearing that would have sufficient area and still be concentric with the reaction.

In Fig. 54 (*a*) is given an illustration of a heel joint for a truss resting directly on a wall. The gusset plate has been carried above the top line of the upper chord, and accommodates an angle clip that distributes the load of the chord throughout the plate, and makes the resistance concentric

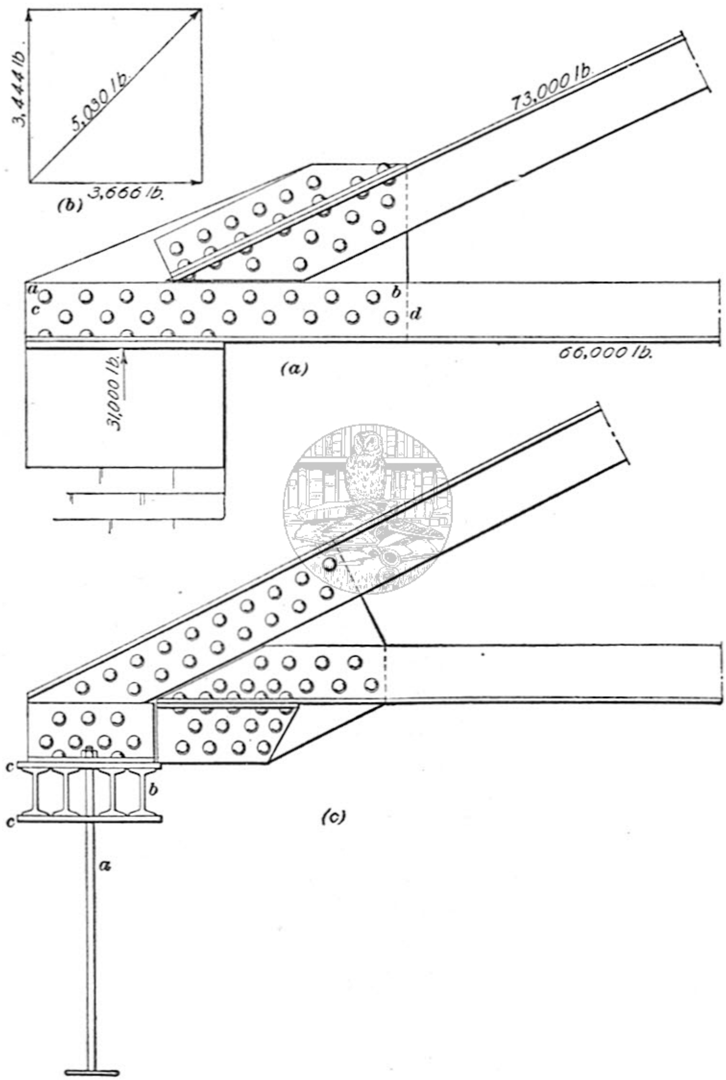

3,444 lb.
5,030 lb.
3,666 lb.
(b)
73,000 lb.
a
b
c
d
66,000 lb.
31,000 lb.
(a)
c
b
c
a
(c)

FIG. 54

with the neutral axis of the chord, the gusset plate being carried out far enough to centralize the bearing. The angles of the tension member are connected to the gusset plate by one leg only, and since in this case the horizontal flanges are used for a bearing on the wall, care must be taken that the stress in the rivets due to the tension in the tie-member and the vertical reactions at the wall bearing combined do not exceed the maximum working stress of the rivets. In calculating the strength of the tension member, it must be remembered that only one leg of the angle may be considered, because the leg on which the bearing plate is riveted has no direct connection to the gusset. The following example illustrates these principles:

EXAMPLE.—If the tension in the lower chord of a steel truss is 66,000 pounds, and the reaction 31,000 pounds, what will be the greatest stress in the rivets directly over the bearing plate connecting the lower chord to the gusset plate, provided that the joint is constructed as in Fig. 54 (*a*)?

SOLUTION.—The eighteen rivets along the lines *a b* and *c d* withstand equal shares of the tensile stress of the lower chord, each providing for $66{,}000 \div 18 = 3{,}666\frac{2}{3}$ lb. The reaction is distributed among the nine rivets located along the line *c d* directly above the plate, giving each $31{,}000 \div 9 = 3{,}444\frac{4}{9}$ lb. to care for. The parallelogram of forces is laid out as in (*b*), and from this diagram each rivet immediately above the plate is found to be subjected to a stress equal to the resultant of the forces of 3,444 and 3,666 lb., which is found to be 5,030 lb. Ans.

If the rivets are $\frac{3}{4}$ inch in diameter and the gusset plate $\frac{3}{8}$ inch thick, they are amply strong, for when an allowable unit shear of 9,000 pounds and an allowable unit bearing of 18,000 pounds are assumed, the safe resistance of a steel rivet is somewhat in excess of 5,030 pounds.

103. In the heel joint in Fig. 54 (*c*) is given another method for securing a concentric bearing on the wall. The gusset plate is extended below the tie-member instead of above the rafter member as in (*a*). Small **I** beams are placed along the top of the wall and form a support for the truss whose bearing is constructed of a plate and angle clips. Additional anchorage is obtained for the truss by

building into the masonry a long bolt a, which passes between the I beams and through the angles and plates. The tension member is connected by both legs to the gusset plate and is figured for its full section, minus the rivet

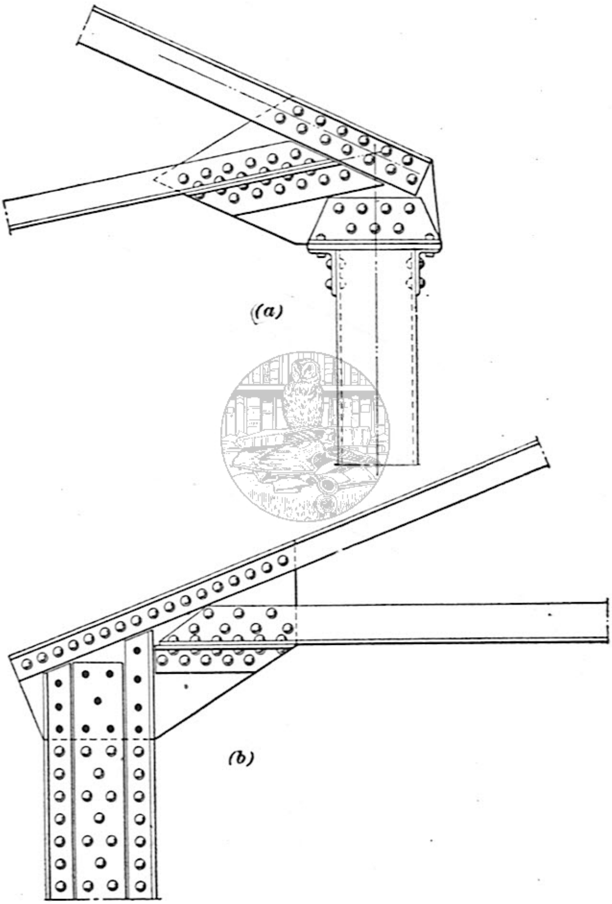


Fig. 55

holes. This design is useful where an uninterrupted line is desirable for the upper chord.

104. In Fig. 55 (a) is given a detail for the heel joint of a truss whose lower chord slants downwards. The truss is

connected to the top of a channel column by means of angle clips and bolts, the whole being arranged so that the center line of the column intersects the panel point. In Fig. 55 (*b*), the truss is connected to a column made of plates and angles. The gusset plate is placed between the angles of the column and connected thereto by means of splice plates, making a compact and rigid joint.

105. Often when a ceiling exhibiting some particular architectural effect is desired, it becomes necessary to carry the bearing below the line of the lower chord. In such cases the design in Fig. 56 is suitable. Here a large gusset

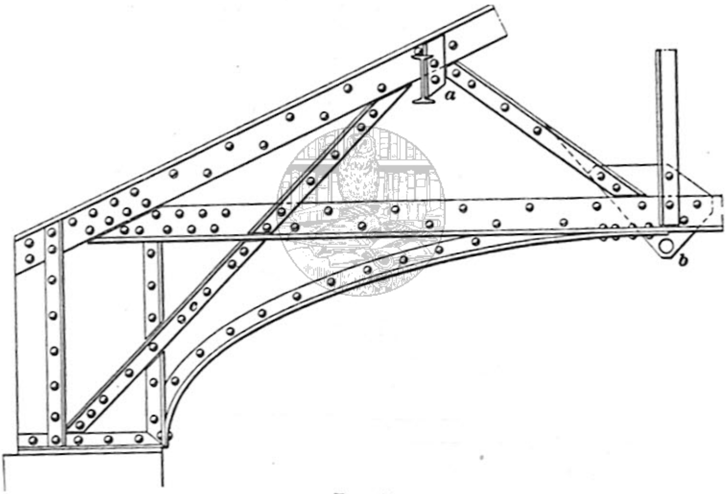


FIG. 56

plate extends almost to the first connection of the lower chord and includes the first panel point of the upper chord, forming practically a small cantilever beam at the end of the truss that would be in action more under an oblique load than under a vertical load. This detail should be analyzed for both shearing and transverse stresses. An angle brace *c* extending from the corner of the bearing to the first panel point in the upper chord greatly strengthens the design. The purlins consist of **I** beams hung by means of angle clips, as at *a*. The truss is designed to carry a

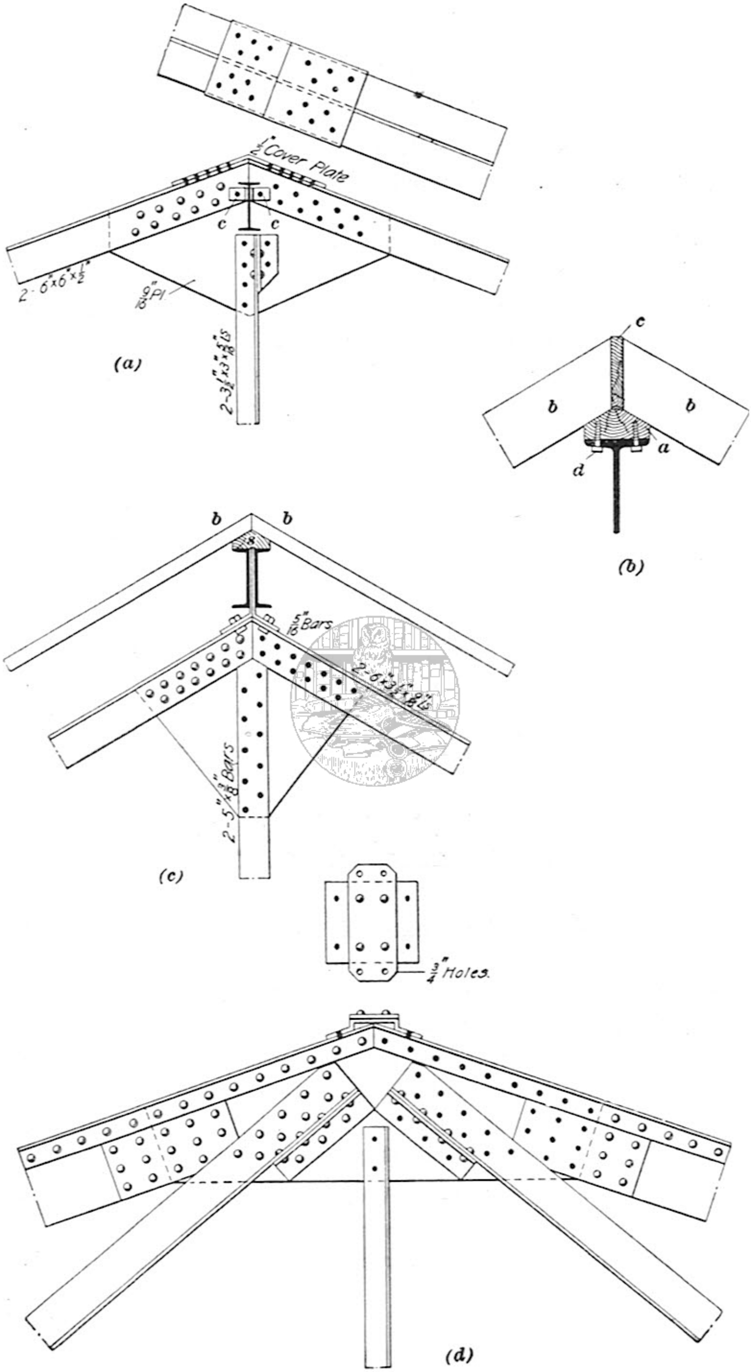

1/2" Cover Plate
c
c
2-6"x6"x1/2"
9/16" Pl.
2-3 1/2"x3"x5/16" Ls
(a)
c
b
b
a
d
(b)
b
b
5/16" Bars
2-5"x3/8" Bars
(c)
3/4" Holes
(d)

FIG. 57

suspended load from the point *b*, a hole being left in the gusset plate to accommodate the suspension rod.

106. Peak Joints.—In Fig. 57 (*a*) is given a detail of a **peak joint** where the members are connected by means of a gusset plate and a cover-plate. An **I** beam serves as a ridge pole, being connected to the rafter members by means of two angle clips *c*, *c*. The rafters are nailed to a strip of wood screwed on top of the **I** beam, as in (*b*). Another detail for a peak joint is shown in (*c*). In this case the tension members are flat bars and the ridge pole is made of two channels riveted back to back and connected to the truss by

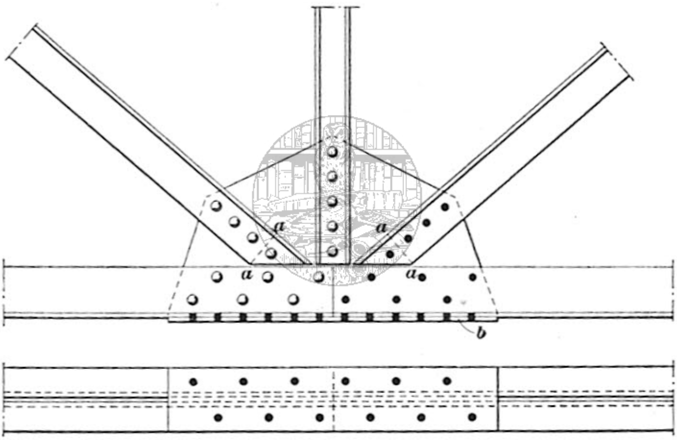


FIG. 58

means of two bent plates extending upwards between them. The roofing boards *b*, *b* are fastened to a nailing strip *s* bolted to the top of the channels. The upper chord of the truss in (*d*) is composed of two angles and a web-plate. In order to preserve the strength of the compression member, its web is connected to the gusset plate by two splice plates. The wooden ridge pole is secured in place by being bolted through holes in the bent plate that supports it.

107. Center Joints.—A very neat design for a **center joint** is given in Fig. 58, in which the gusset plate is used as a splice plate for the lower chord. In structural work the

angles are usually cut square on the ends, as shown by the dotted lines a, a, although greater neatness may be secured by cutting them obliquely, as in the figure. Such a center joint as this could be used in a structural truss constructed along the lines of a Howe truss, where the compression members are oblique and the tension members vertical. The center joints that serve also as splices are usually those at which the truss has been separated for convenience in shipment. Both field and shop rivets are used in this design, the gusset plate being riveted to the left-hand portion of the truss in the shop, and sent with that portion.

The center joint, when combined with a splice, should always be reenforced by a flange splice plate, as at b. Such a plate increases the strength of the splice and stiffens the truss laterally at a point that would otherwise be weak.

EXPANSION ENDS

108. When trusses of long span are exposed to great variations of temperature, provision is usually made for the expansion and contraction of the frame. This may be done by the use of roller bearings, flat plates sliding on each other, or by a swinging column or arm placed at the end of the truss.

Since flat plates are not efficient they are only used in trusses of short span whose contraction and expansion are but slight. As they are not without friction, allowance is sometimes made for a horizontal force, which is found by the following formula:

$$H = R\,a \qquad (6)$$

in which H = horizontal force to be introduced at expansion end of truss;

R = reaction on free end of truss;

a = coefficient of friction for metal sliding on metal.

For steel on steel or wrought iron on wrought iron a may be taken at .15, while for wrought iron or steel on cast iron a is .20.

109. In Fig. 59 is illustrated the **expansion end** commonly employed for a steel truss having a span of 60 or 80 feet. A sole plate $\frac{7}{16}$ to $\frac{7}{8}$ inch in thickness is provided at the expansion end and is incorporated in the heel joint, being secured to the angle clips with countersunk rivets, as at *a*.

An illustration of the common type of roller expansion joint is given in Fig. 60 (*a*). In this case the truss is supported on a bearing plate resting on six rollers each 2 inches in diameter and 13 inches long. The rollers are held in place by a bar-iron frame and kept the proper distance apart by separators made of pipe, through which bolts are passed

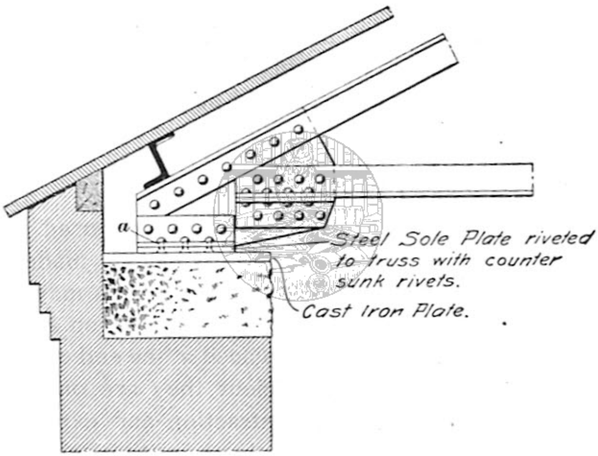


FIG. 59

and riveted over on the ends, as shown in (*b*). The truss end is strengthened by riveting a cover-plate over both angles and splice plate, as at *a* in (*a*).

Such a roller bearing as that shown in Fig. 60 (*c*) is used only for heavy trusses with spans of 80 feet or more. It is not customary to hinge a supporting column at the top when it connects with a steel truss, but structural engineers often use a combination roller-and-hinge bearing for long trusses and heavy work. In this construction, shown in Fig. 60 (*c*), a carriage or roller-bearing supports the expansion end of the truss by means of the pin connection. By the use of

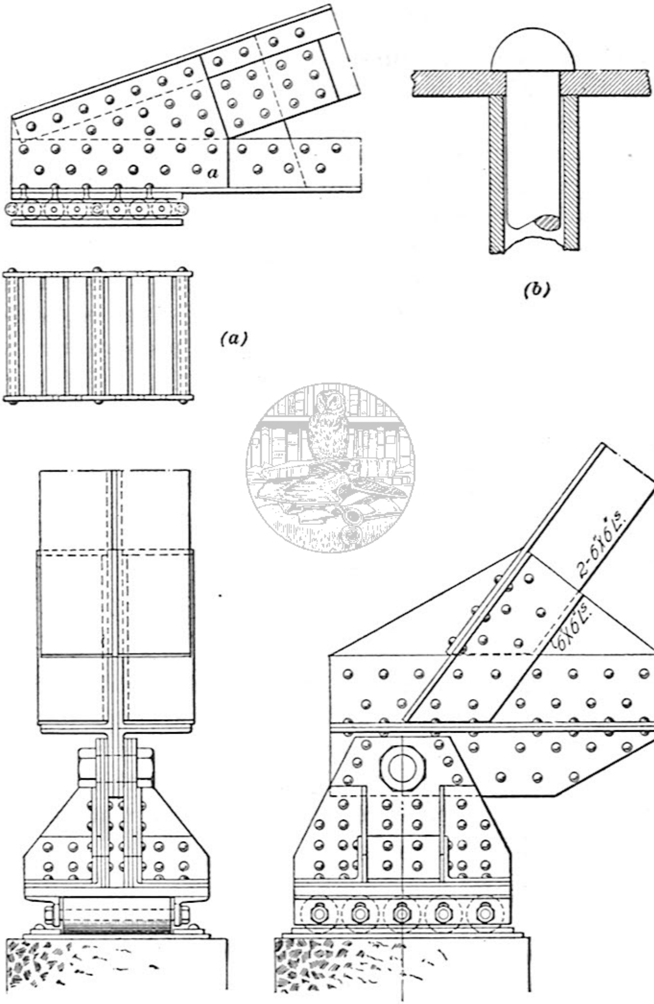


Fig. 60

such a bearing the deflection of the frame does not cause an unequal pressure on the bearing, and a consequent overloading of some of the rollers.

110. Roller Bearings.—There is very little definitely known from which the strength of roller bearings may be conclusively determined. However, at Cornell University an extensive series of experiments was conducted, using rollers varying from 1 inch to 4 inches in diameter, rollers and plates being made of wrought iron, cast iron, and steel. As the results gained from these experiments differ materially from the theoretical analysis, an empirical formula was devised, which approximated the results obtained, and by which a roller-expansion bearing may be proportioned with an assurance of safety.

Let p = allowable load, in pounds per lineal inch of roller;
D = diameter of roller, in inches;
P = total load to be carried;
l = length of roller.

Then
$$p = 1,200\sqrt{D} \qquad (7)$$
and
$$l = \frac{P}{p} = \frac{P}{1,200\sqrt{D}} \qquad (8)$$

This assumes that the load is uniformly distributed over the length of the rollers, and embodies a factor of safety of 5, if rollers and bearing plates are of cast iron, and 3, if of wrought iron or steel. By the use of this factor of safety, the elastic limit is regarded as the ultimate strength of the material.

111. If the machine work is poor and the roller not liable to bear on its full length, the formulas given below are recommended. These embody a factor of safety twice as great as in the above formulas, making the factor for cast iron 10, and for wrought iron and steel 6.

$$p = 600\sqrt{D} \qquad (9)$$

$$l = \frac{P}{600\sqrt{D}} \qquad (10)$$

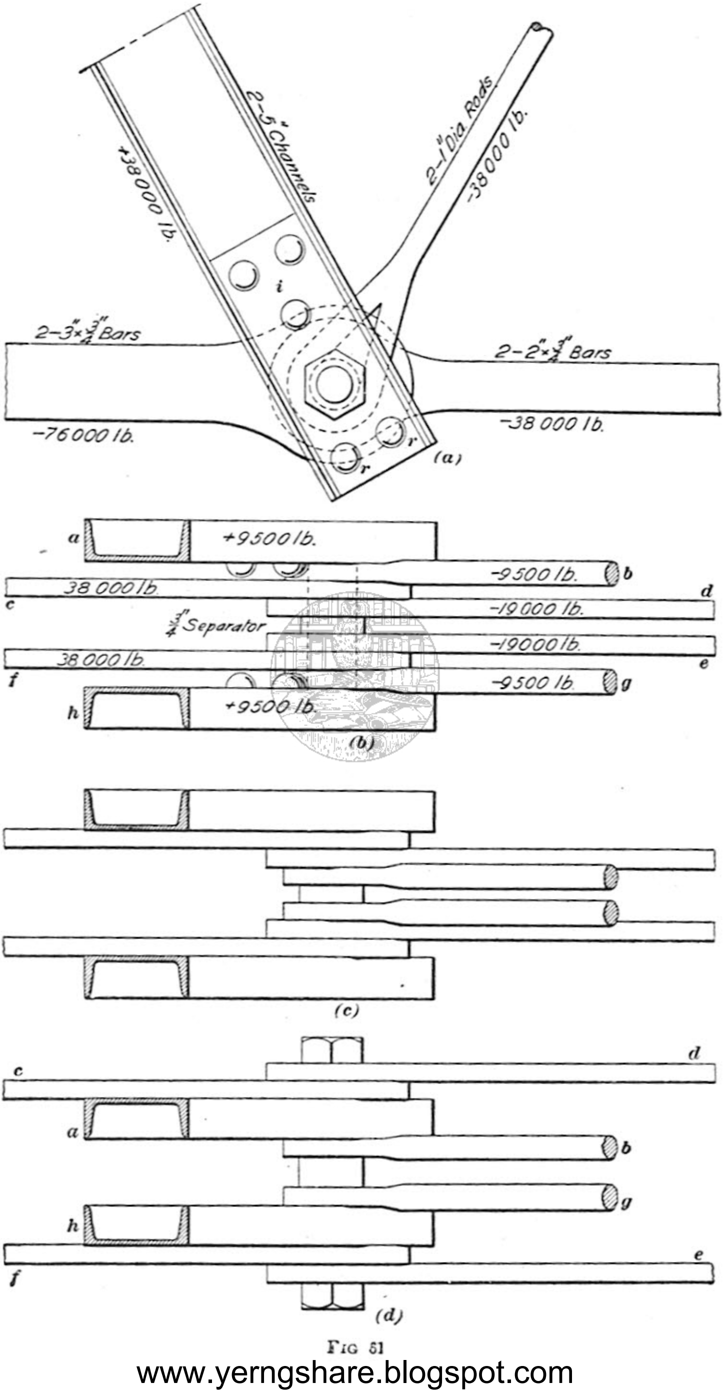
EXAMPLE.—If the reaction at the free end of a truss is equal to 90,000 pounds, how many rollers 3 inches in diameter and not more than 9 inches long will be required to support the truss?

SOLUTION.—Using formula **7**, a 3-in. roller will support a load of $1,200\sqrt{3}$, or $1,200 \times 1.732 = 2,078$ lb. $90,000 \div 2,078 = 43+$ in. Hence, it would be well to use five rollers 9 in. long, or six rollers 8 in. long. Ans.

112. Pin-Connected Joints.—In the design of a **pin-connected joint** the size of the pin must be found and the bearing of the metal around the pinhole investigated. After the arrangement of the member has been determined, a pin is selected that must be large enough to resist the maximum bending moment coming on it. The shear of the pin and the bearing of the plate or member must be taken into consideration also, so that the safe unit stress is not exceeded. Often, while a pin of given diameter can safely resist the maximum bending moment imposed, when the unit bearing value of one of the bars resting on it is calculated it is found that the bearing area of the pin is insufficient, so that a pin of larger size must be used, as in many cases the thickness of the bars cannot be increased.

The number of methods of connecting the different members to the pin is unlimited, but the following are the ones usually employed.

113. In Fig. 61 (*a*) is given a detail of a lower chord joint that is used in a Fink truss of heavy design. All the tension members are in pairs, as shown in the plan of the connection (*b*). The compression member, composed of 5-inch channels, is strengthened at the pinhole by pin plates riveted between the flanges of the channels. Since the web of the channel is less than $\frac{7}{16}$ inch in thickness, no countersunk rivets should be used without employing an inside plate as well, and as it is desired to avoid the use of another plate the rivets must be so arranged that they will not interfere with the members placed next to the strut. Both the pin plate and the channel should be extended far enough beyond the pin to allow two rivets r, r to be driven; this adds strength to the pin plate.



+38000 lb.
2-5" Channels
2-1" Dia. Rods
-38000 lb.
i
2-3"×3/4" Bars
2-2"×3/4" Bars
-76000 lb.
-38000 lb.
r
r
(a)
a
+9500 lb.
-9500 lb.
b
38000 lb.
c
d
-19000 lb.
3/4" Separator
-19000 lb.
38000 lb.
e
f
-9500 lb.
g
h
+9500 lb.
(b)
(c)
c
d
a
b
g
h
f
e
(d)

FIG 51

In Fig. 61 (*b*) is shown the method of arranging the several members on the pin so as to obtain the least bending moment. The two rods *b*, *g* tending to raise the pin are placed next to the strut members *a*, *h*, which tend to lower it. The tension in the two bars *c*, *f* is balanced by the tension in the rods *d*, *e*, and by the horizontal components of the stress in the members *a*, *b*, *g*, and *h*. Since the horizontal component of the stress in the members *g*, *h* is 19,000 pounds, by placing these on one side of the member *f* and placing *e* on the other side, the balance on the pin is almost perfect.

But frequently the size of the pin is not so important as it is to secure such an arrangement that the greatest possible lateral stiffness is secured. When this is the case, the members in the lower chord should be placed as far apart as possible, as shown in Fig. 61 (*c*). When all things are considered and each point given its relative importance it is found that the arrangement in (*d*) is the best that can be adopted. In this case the greatest lateral stiffness is obtained, without materially increasing the bending moment on the pin.

114. The detail shown in Fig. 62 may be adopted when it is necessary to splice the upper chord. The strut is made of two channels placed back to back, and turned so that their webs are at right angles to the plane of the truss. The pin plates are riveted to the flanges of the channels and extend up between the upper chord member, thus serving as splice plates also. These pin plates are shop-riveted to one portion of the chord member and field-riveted to the other part, and should be of such a thickness that the bearing on the pin is reduced to the allowable unit stress. The end of one of the tension rods is provided with a clevis, and the end of the other with a loop that is placed between the sides of the clevis; by this means the rods are not only connected to the pin, but are also kept in the same vertical plane.

115. The details given in Fig. 63 can be used in very large pin-connected trusses. The truss proper ends at the joints *a*, *b*, but in order to stiffen the roof against the wind a brace is extended from *a* to *c*, which increases the rigidity of

the frame. This brace must be designed to resist both tension and compression, since its stress varies as the wind shifts from one side to the other. The upper chord member is composed of two $16'' \times \frac{1}{2}''$ web-plates to which four $3'' \times 3'' \times \frac{1}{2}''$ angles are fastened and held together by lacings running along both top and bottom of the member. Latticed beams placed centrally with the panel points, as at *d*, serve as purlins, carrying the 6-inch $14\frac{3}{4}$-pound channel rafters. The trusses are fastened to columns composed of two 15-inch

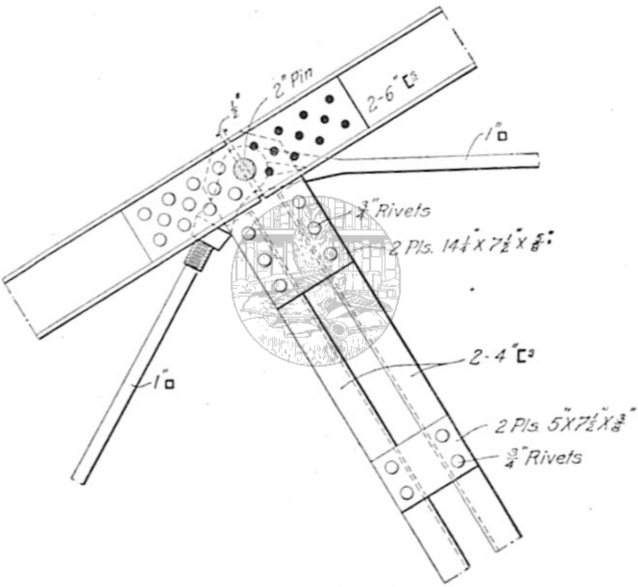


FIG. 62

42-pound **I** beams riveted to an 18-inch 55-pound **I** beam. This makes a column amply strong for the position and at the same time very simple in construction. These details are taken from the design for the Rock Island terminal station in Chicago.

116. The analysis and proportioning of an important pin connection in a roof truss are best demonstrated by considering the design of the connection and members of a joint such as is given in Fig. 64. *A B*, *B C*, *C D*, and *D A*, shown in the

frame diagram (*a*), represent members whose lengths are 23, 20, 15, and 15 feet, respectively. In the solution, all tension members are assumed to be composed of bars 5 inches

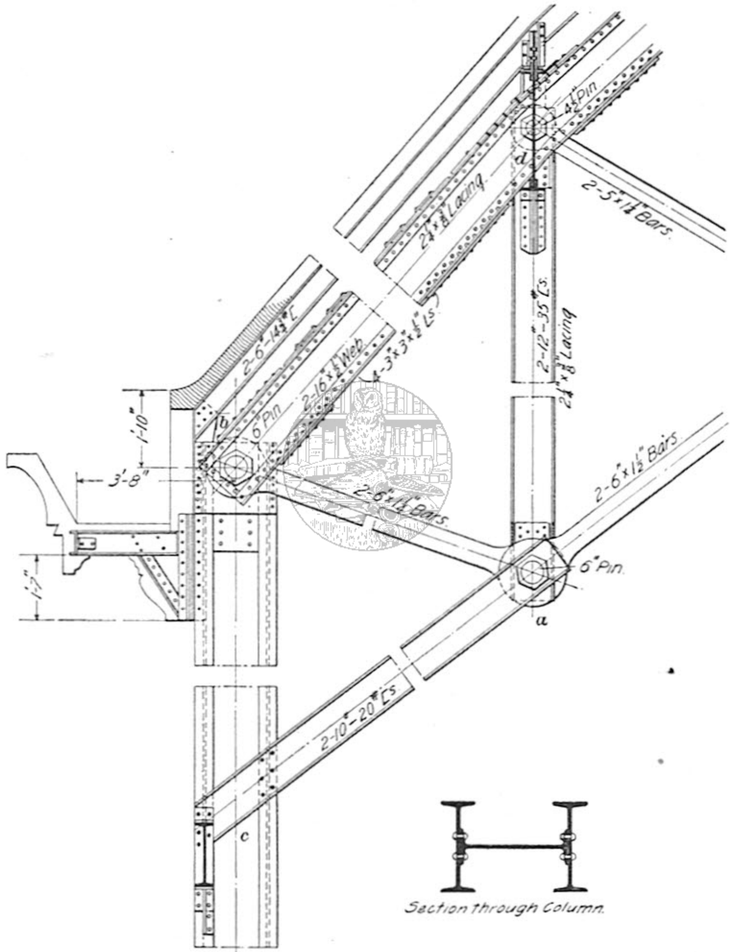


Fig. 63

deep, having a maximum allowable unit tensile strength of 18,000 pounds, while the compression member is to be constructed of channels placed back to back and latticed together.

In proceeding with the problem, it is first necessary to lay out a skeleton diagram, as indicated in (*b*) by the dot-and-dash lines, on which the design is drawn after the sizes of the different members have been determined. After this is done the stresses in the several members may be either obtained from the stress diagram or calculated by some mathematical process. In this case, the compression member BC must sustain 78,000 pounds, and if the column is composed of two 9-inch 15-pound channels, their combined area will be 8.82 square inches and the radius of gyration of the column section about an axis perpendicular to the web is 3.4. Applying the formula $u = \dfrac{50{,}000}{1 + \dfrac{l^2}{18{,}000\, r^2}}$, in which u equals the ultimate strength in pounds per square inch of section for a column hinged at the ends and built of medium steel, the calculations may be made as follows:

$$u = \frac{50{,}000}{1 + \dfrac{(240)^2}{18{,}000 \times (3.4)^2}} = 39{,}160 \text{ pounds}$$

If a factor of safety of 4 is assumed, the safe unit stress that the column or strut will sustain is $39{,}160 \div 4 = 9{,}790$ pounds. Hence, if the area of the column or strut section is 8.82 square inches, the safe sustaining power of the strut is $9{,}790 \times 8.82 = 86{,}348$ pounds. But since the member is required to sustain a load of but 78,000 pounds, it is found to be large enough and the section originally assumed may be used.

117. In proportioning the tension members in the example under discussion, it is advisable to investigate the effect of the weight of the member and to determine the unit stress to which the member will be subjected on account of the deflection produced by its own weight. The additional stress to which the member is subjected by the transverse stress produced by its weight may be determined by formula **1**.

Applying this formula for a steel tension bar of the

assumed width and depth of 1 inch and 5 inches, respectively, and a length of 15 feet, or 180 inches,

$$s_1 = \frac{M_2 c}{I + \frac{W_t l^2}{10 E}} = \frac{\left(.28\,b\,d\,l \times \frac{l}{8}\right) \times c}{\frac{b\,d^3}{12} + \frac{W_t l^2}{10 \times 28{,}000{,}000}}$$

and, by substitution, after W_t has been assumed to be equal to the strength of a 1-inch × 5-inch bar with a unit stress due to the direct tension of 17,000 pounds per square inch, or a total stress of 85,000 pounds, the value

$$s_1 = \frac{.28 \times 1 \times 5 \times 180 \times \frac{180}{8} \times \frac{5}{2}}{\frac{1 \times 5 \times 5 \times 5}{12} + \frac{85{,}000 \times 180 \times 180}{10 \times 28{,}000{,}000}} = 700 \text{ lb., approx.}$$

If the allowable direct tensile stress is 17,000 pounds, the addition of the unit stress of 700 pounds due to the transverse stress created by the weight of the bar, the total unit stress on the bar, is 17,700 pounds. As steel bars used in such structures as roof trusses can be depended on to withstand a unit stress of 18,000 pounds, it is evident that the safety of the bar is not endangered by its weight, and that a unit tensile stress of at least 17,000 pounds may be assumed in proportioning the tension bars; in fact, a unit resistance of 18,000 − 700 = 17,300 pounds may be considered as the allowable unit resistance of the bars to tension. Tension members composed of bars should be proportioned to withstand the maximum stress to which they will be subjected, and should be assumed to be of some uniform width that will permit the formation of suitable eyes at the ends of the bars. After deciding what width is to be used, the total width of bar required to resist the stress may be found by dividing the maximum stress in the member by the allowable stress for a bar of unit width, as just determined, after allowing for the stress created by the weight of the bar itself. For instance, the stress in the member CD, Fig. 64 (a), is 433,280 pounds, so that if a bar 1 inch in thickness and 5 inches deep will sustain 5 × 17,300 = 86,500 pounds, to withstand 433,280 pounds, a bar 433,280 ÷ 86,500, or practically 5 inches

in width will be required. Hence, if four bars are used for this member each will have a depth of 5 inches and a width of $5 \div 4 = 1\frac{1}{4}$ inches. As the rods DA are subjected to a tensile stress of 310,000 pounds, a combined width of bars of $310,000 \div 86,500 = 3.58$ inches will be required, and if this is the width for four bars, each should be about $\frac{15}{16}$ inch wide, and 1 inch in thickness. In the same manner it may be found that two $1\frac{1}{16}$-inch bars will be ample for the member AB.

118. After finding the dimensions of the several members meeting at the joint, the size of the pin required and the arrangement of the different members on the pin must be determined. From a theoretical standpoint, it is best to place members that supply opposing forces as near each other as possible, in order to reduce the bending moment. To produce equilibrium on the pin, the force CD must be equal to the algebraic sum of the horizontal components of AB and DA, and BC must be equal to the algebraic sum of the vertical components of AB and DA. By the method of resolution of forces, AB and DA are found to have horizontal components of 63.64 tons and 153 tons, respectively, and vertical components of 63.64 tons and 24.64 tons, respectively, and as the algebraic sum of the horizontal forces is $153 + 63.64 = 216.64$ tons, and the algebraic sum of the vertical components is $63.64 - 24.64 = 39$ tons, the pin is in equilibrium.

Now, if a combination of opposing forces can be found by which the forces in each of the four bars CD are taken up directly by one or two of the bars in the members AB or DA, the bending moment will be greatly reduced. A good arrangement is shown in Fig. 64 (*c*), in which *No. 1* DA is placed on the outside, then *No. 1* CD, and next the vertical strut *No. 1* BC, which gives the required width between the channels. These are followed by *No. 1* AB, *No. 2* CD, and *No. 2* DA in the order mentioned, so that CD is placed between the ties DA and AB that oppose it, making a practically balanced load on the pin. The two central members DA are separated by a 1-inch washer, which holds all

the members tightly on the one pin and facilitates painting. All the members on each side of the washer should be placed as close together as possible; to accomplish this, countersunk rivets are required in the pin plates on the strut, but as the web is too thin to permit countersinking, a $\frac{7}{16}$-inch plate is required on each side of the web, the flange being cut to allow these tension members to set closely to it.

Fig. 64 (*d*) is a general diagram showing the location and amounts of the forces that act on one-half the pin, while (*e*) and (*f*) are the diagrams for the horizontal and vertical forces, respectively, giving the amount and location of each and also their direction with respect to each other. The forces *CD* oppose the horizontal forces *DA* and *AB*, as shown in (*e*), while in (*f*) the vertical force *AB* opposes the forces *DA* and *BC*. It will be noted in both (*e*) and (*f*) that the forces on one side of the pin exactly balance those on the other side.

119. The horizontal and vertical bending moments of the various forces must now be found; the maximum bending moment can then be readily obtained, since in each case it is equal to $\sqrt{M_h^2 + M_v^2}$, in which M_h represents the horizontal bending moment and M_v the vertical. When there are but two forces this is readily accomplished, but when there is a number of forces meeting at a point, it is well to use a graphical method of calculation.

In Fig. 64 (*g*) is drawn a diagram representing the resistance of the pin to the horizontal components of the forces, only one-half of the pin being shown. Beginning at the left, a line *a b* is laid off representing $\frac{15}{16}$ inch, the thickness of *No. 1 D A*; then *bc*, representing the thickness of *No. 1 C D*, $1\frac{1}{4}$ inches; *c d* representing the thickness of the member *B C*, or .29 inch for the channel and .875 inch for the two $\frac{7}{16}$-inch plates used, making a total of 1.165 inches. Continuing, *de*, *ef*, and *fg* are laid off to represent the thickness of *No. 1 A B*, *No. 2 C D*, and *No. 2 D A*, or $1\frac{1}{16}$ inches, $1\frac{1}{4}$ inches, and $\frac{15}{16}$ inch, respectively.

Before completing (*g*) the diagram in (*h*) is drawn. The load line is first drawn beginning at *a*; *a b* represents the magnitude of *No. 1 D A*, as shown in (*e*), or 38.25 tons; *b c*, *No. 1 C D*, or 54.16 tons. The load next in order on the pin is *B C*, but as this is a vertical load it is not considered in the diagram for the horizontal forces, and hence the points *c* and *d* coincide. Then from *d*, *d e* is drawn representing *A B*, or 31.82 tons; *e f*, *No. 2 C D*, or 54.16 tons; and last *f g*, *No. 2 D A*, or 38.25 tons. From *a*, the middle point of *b f*, a horizontal line *o a* is drawn and some value assumed for it, as 50 tons; then the point *o* is connected with the various points on the line *b f*. Reverting to the diagram (*g*), *a 1* is drawn parallel to *o a*. A line from *1* is drawn parallel to *o b* and prolonged until it intersects a vertical line drawn through the middle of *b c*, at some point *2*; from *2*, a line is drawn parallel to *o c* and would intersect a vertical line through the middle of *c d* at some point *3*, but since *c d* represents the width of *B C*, a vertical member, it is not considered and the line *2*–*3* is prolonged until it intersects a vertical line through the middle of *d e* at *4*. From *4*, a line is drawn parallel to *o e* and intersects the vertical line drawn through the middle of *e f* at *5*. From *5*, a line is drawn parallel to *o f* and prolonged until it intersects a vertical line through the middle of *f g* at some point *6*; and from *6*, a horizontal line is drawn until it intersects the center line of the pin *x y* at 7. Now in each case the horizontal bending moment of any member is represented graphically by the amount the broken line deviates from the horizontal at the middle of the line representing the thickness of that member, and the greater the deviation the greater is the bending moment. From this it is evident that the greatest bending moment is at the point *2*. The intensity of this bending moment is found by multiplying the amount of this deviation by the polar distance *o a*, giving a result of $.85 \times 50 = 42.5$ inch-tons, or 85,000 inch-pounds, for the point *2*, and $.51 \times 50 \times 2{,}000 = 51{,}000$ inch-pounds for the point *5*.

The vertical diagram is laid out in practically the same

way, and on the load line, $a\,b$, $c\,d$, $d\,e$, and $f\,g$, represent the amounts and directions of *No. 1 D A*, *No. 1 B C*, *No. 1 A B*, and *No. 2 D A*, respectively. The diagrams (*i*) and (*j*) are drawn similar to (*g*) and (*h*) and the deviations of the broken line from the horizontal at the points *2* and *5* are found to be .15 inch and .73 inch, respectively, making the bending moments 15,000 inch-pounds and 73,000 inch-pounds, respectively. Then, the maximum bending moment at the point *2* equals $\sqrt{M_h^2 + M_v^2}$ or $\sqrt{(85{,}000)^2 + (15{,}000)^2}$ $= 86{,}313$ inch-pounds, and at point *5*, $\sqrt{(51{,}000)^2 + (73{,}000)^2}$ $= 89{,}050$ inch-pounds. The latter is therefore the maximum bending moment on the pin, and the size of the pin may now readily be determined by substituting in the formula $M = \frac{S\,I}{c}$. Assuming the maximum fiber stress as 18,000 pounds and substituting $.0982\,d^3$ for $\frac{I}{c}$, $89{,}050 = 18{,}000 \times .0982\,d^3$, or $d^3 = \frac{89{,}050}{1{,}767.6} = 50.38$, and $d = \sqrt[3]{50.38} = 3.693$ inches. A $3\frac{11}{16}$-inch pin may be used with safety since its diameter, 3.6875, is only a trifle less than 3.693 inches.

120. Frequently the shear and bearing value are of as much importance as the maximum bending moment in determining the size of the pin, and hence must be investigated. Assuming a shearing value of 9,000 pounds per square inch, the total shearing value of the pin is 9,000 times the area of the pin or $9{,}000 \times .7854 \times (3\frac{11}{16})^2$, or about 96,120 pounds.

The shear is determined by drawing the horizontal and vertical shear diagrams, as shown in Fig. 64 (*k*) and (*l*).

In the horizontal shear diagram (*k*), a horizontal line $h\,n$ is drawn, and on it the points o, x, r, e', and f' are marked to correspond with b, c, d, e, and f in (*g*). Then through these points lines perpendicular to $h\,n$ are drawn, on which are laid off distances to represent the stress of the various members. Thus, $o\,i$ represents 38.25 tons, the stress in *No. 1 D A*; $y\,j$ represents 54.16 tons, the stress in *No. 1 C D*; $z\,l$ the stress in *No. 1 A B*; $t\,m$ the stress in *No. 2 C D*; and $u\,n$

the stress in *No. 2 D A*. The line jk is parallel to hn since BC is a vertical member, and this diagram is for horizontal shear. In the same manner, the vertical shear for each member may be found as in (l).

In order to find the maximum shear at any point, a graphical method may be applied with advantage. Thus, the horizontal shear at the point b, Fig. 64 (g), is equal to oi as shown in (k), and the vertical shear for the same point is equal to oi in (l). Then if, in (k), op is laid off equal to oi in (l), and the line ip is drawn, this line will represent the maximum shear, as it is in reality the square root of the sum of the squares of the horizontal and vertical shears, respectively. In the same way, by laying off rs in (k) equal to rk in (l), and joining the points k and s, ks represents the maximum shear at (d). It can readily be seen that ip is greater than ks, hence ip represents the maximum shear and by scaling is found to be about 39 tons, or 78,000 pounds, approximately. From this it is evident that the pin is sufficiently strong in shear.

The bearing value must next be considered. The allowable tension on the bars, and consequently the bearing per lineal foot on the pin, is 86,500 pounds; hence, if the allowable bearing on the pin per square inch of section is 20,000 pounds, the required diameter will be $86{,}500 \div 20{,}000 = 4.325$ inches; hence, a $4\frac{3}{8}$-inch pin may be used.

121. Pin Plate for Channel.—The total strength of the channel column is 86,348 pounds, and since the allowable pressure per square inch on the pin is 20,000 pounds, the required area on one side will be $\frac{86{,}348}{2 \times 20{,}000}$, or about 2.16 square inches. The total bearing of the strut on the pin is equal to the product of the diameter of the pin by the combined thickness of the pin plate and channel. This thickness for one member is equal to the sum of the width of the channel web and the two $\frac{7}{16}$-inch plates, or $.29 + 2 \times .4375 = 1.165$, and for the two members is 2.33 inches. Hence, the bearing area equals 4.375×2.33, or about 10 inches, which

is ample. Then the bearing per square inch is equal to $\frac{86{,}400}{2.33}$, or 37,081 pounds, and for a $\frac{7}{16}$-inch plate would be $\frac{7}{16} \times 37{,}081$, or about 16,200. Assuming the bearing value at 12,000 pounds per square inch, the value of a $\frac{3}{4}$-inch rivet in single shear, when the plate is $\frac{7}{16}$-inch thick, is 3,940. Hence, $16{,}200 \div 3{,}940$, or 5 rivets will be required, but to make the design symmetrical seven rivets have been used, as shown. The pin plates extend 6 inches beyond the center line of the hole and are made as wide as the channel flanges will permit. To bind the whole together two rivets are placed through these plates beyond the hole.

The required outside diameter of the eyes of the tension bars is $4.375 + 1.4 \times 5 = 11.375$ inches. From these dimensions the figure may be completed as shown.